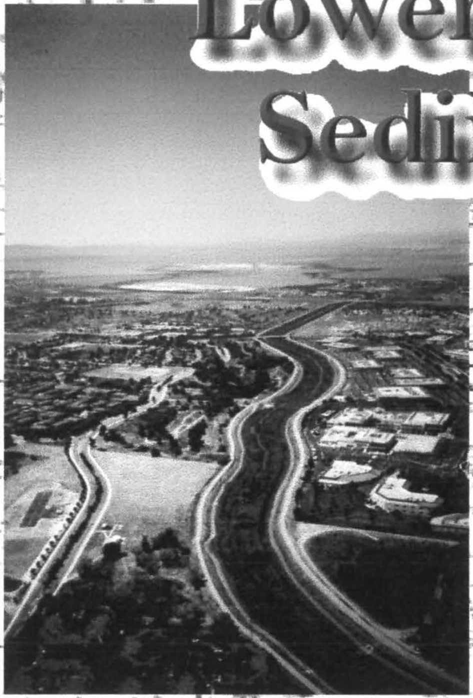
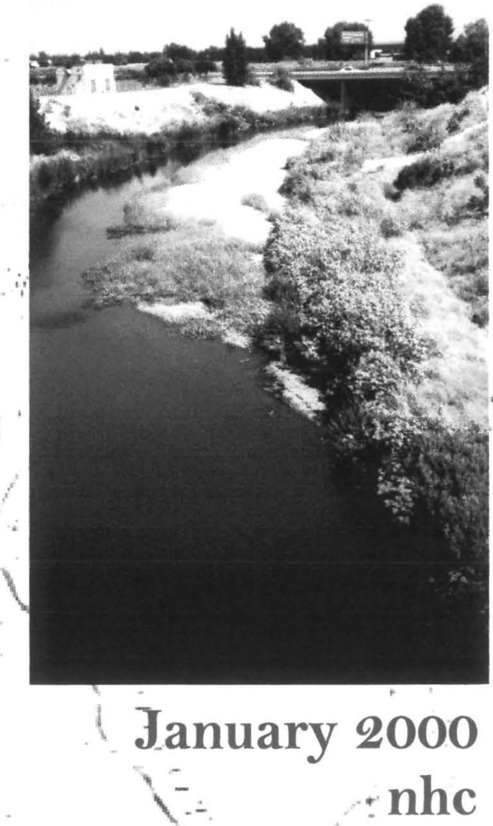
Santa Clara Valley Water District

Final Report

Lower Guadalupe River Sedimentation Study





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Santa Clara Valley Water District
Final Report

Lower Guadalupe River Sedimentation Study

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January 2000

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Chapter 1

Executive Summary

1. Executive Summary

The Lower Guadalupe River Project Area does not have the ability to convey the flow of the 100-year flood event. Located between the San Francisco Bay and Downtown San Jose, the Lower Guadalupe River Project area is subject to many natural and man-induced agents and natural physical processes that affect the hydraulic and sediment transport characteristics of the River. The most significant agents and processes include the local geology, regional seismicity, tidal processes, past subsidence in the region, rising sea levels, changes in land use in the watershed and on the floodplains, channel improvement projects upstream of the Project Area, sediment accumulation from tidal and terrestrial sources, bridge constrictions and aggressive encroachment of in-channel vegetation along portions of the Guadalupe River. All of these agents and processes contribute to decreasing channel capacity with time and to increasing annual maintenance requirements to meet present and future levels of flood conveyance required by the Santa Clara Valley Water District (District).

The purpose of this study is to develop knowledge of the Lower Guadalupe River hydraulics with respect to its ability to convey flow and sediment materials now and in the future. The object is to gather the necessary historical and field information required to develop engineering (quantitative) tools, including a sediment transport model of the Lower Guadalupe River, and to use these to evaluate present and future sediment yield of the study reach, the channel capacity, and maintenance requirements to meet planning, environmental, and design needs of the District. Both long term trends (on the order of decades) of river degradation and aggradation, as well as episodic single flood events (days), are evaluated for both present (existing) conditions and for two alternative channel designs.

Enumerate data show significant reductions in channel capacity since construction of the flood control project. The four primary contributors to the reduction in channel capacity are: (1) watershed derived sediment deposition, (2) tidally influenced sediment movement and accumulation in the lower reaches, (3) growth of vegetation and accumulation of debris within the project levees, and (4) bridge constrictions. The Lower Guadalupe River Project area is a zone of net sediment accumulation. Sediment deposition is likely to continue in the Project area under most project design and maintenance scenarios. Under existing conditions, long term (70 years from present) sediment accumulation may increase the 100-year water surface elevation by approximately 7.5 feet in the vicinity of Montague Expressway, 6.5 feet in the vicinity of Trimble Road, and 5 feet near Highway 101. For conditions described in the District's alternative 2, the long-term 100-year water surface elevation at Montague Expressway increases by approximately 10.5 feet. Immediately downstream of I-880, the channel is steeper and is experiencing channel incision. This is likely to continue unless its arrested with scour counter-measures. The majority of the coarse gravels transported to the Project reach from upstream are deposited in the vicinity of Airport Parkway to Trimble Road, while watershed derived silts and clays deposit in the tidal reaches during low flow periods. Enhanced by a steadily rising sea level, tidal accretion of Bay muds will continue to raise the channel invert and marginal marsh surfaces within the leveed reaches affected by tides. Detailed assessments of the potential for increasing flood risks to reaches downstream and appropriate mitigating measures of the Project are required if capacity for 100-year floods is to be maintained.

Chapter 2

Introduction

2. Introduction

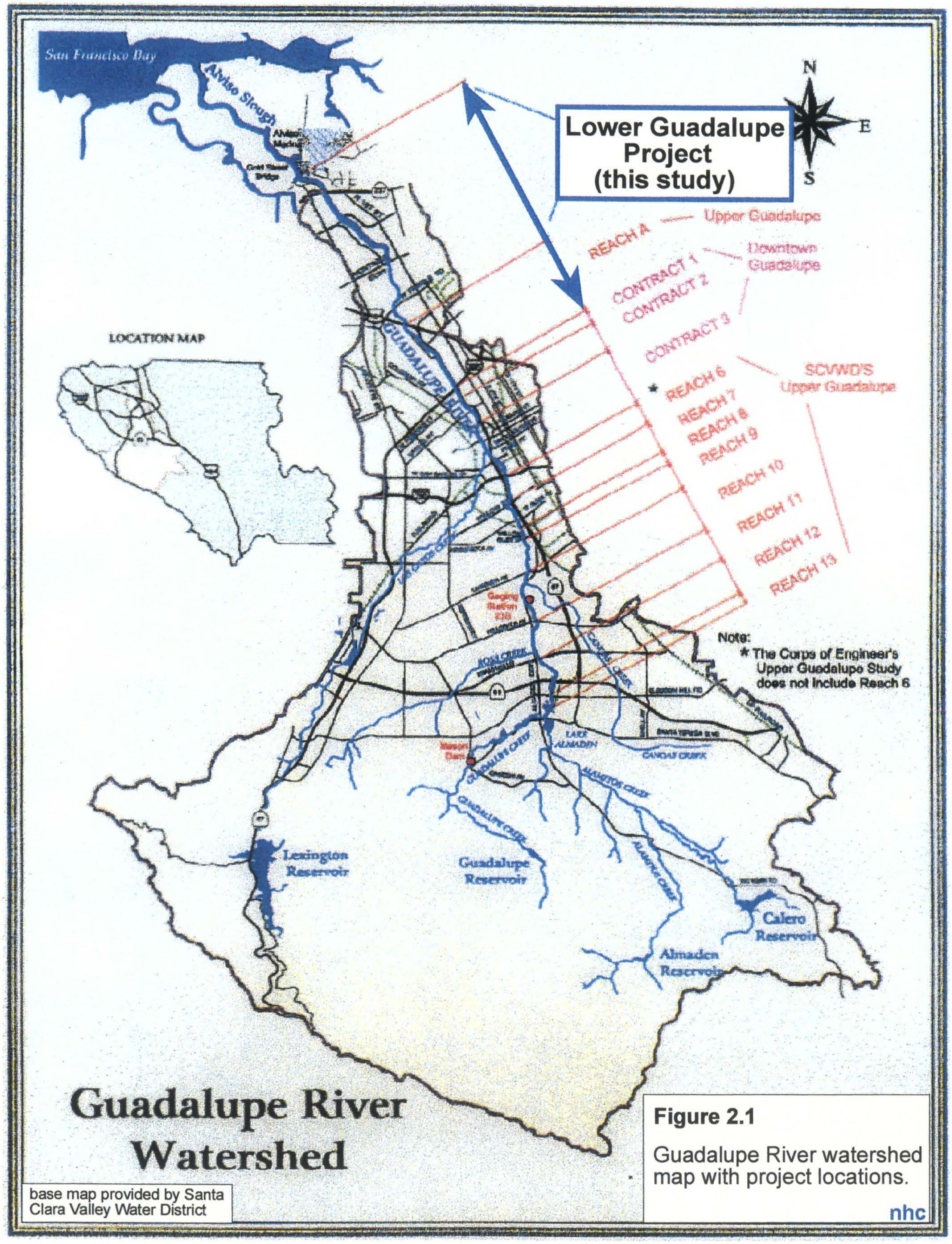
2.1. Setting and Problem Statement

The study area considered during this investigation extends along the Lower Guadalupe River from Highway 880 downstream to the SPRR bridge just south of Alviso, CA (Figure 2.1). This region of the Guadalupe River is referred to as The Lower Guadalupe River Project Area (the Project). This portion of the Guadalupe River does not have the ability to convey the 100-year flood event. Located between the Bay and Downtown San Jose, the Project reach receives runoff from a highly urbanized region consisting of a 160 square mile drainage basin comprised of a steep upper watershed, a highly densified urban residential and light commercial zone (the Upper Guadalupe River), and a significantly developed and encroaching "Downtown" commercial zone. Twelve bridges cross the Lower Guadalupe River within the Project Area; several of the bridges will experience pressure flow conditions during the design event. Refer to the study area map of the Lower Guadalupe River presented in Figure 2.2. Local storm drainage from these areas as well as the Project area is discharged into Guadalupe River adding to the runoff volume. The Upper Guadalupe reach is being evaluated for flood control opportunities by the San Francisco District Corps of Engineers (CESPN). The Downtown reach flood control project is being evaluated, designed and constructed by the Sacramento District Corps of Engineers (CESPK). Portions of the Lower Guadalupe River have been evaluated previously by the Santa Clara Valley Water District (District) and the CESPK with some constructed improvements (flood control levees) down to the County Marina just north of the Town of Alviso. Portions of these levees have had to be raised at least twice since 1960 to reduce increasing flood risks due to inadequate channel capacity. Lands downstream of the SPRR bridge are for the most part privately owned and maintained. No other major flood control facilities currently exist throughout the Project area. Although reservoirs exist on all the major tributaries upstream of the Project, these facilities are operated primarily for water supply, not flood control.

A number of natural and man-induced agents and physical processes affect sediment transport and flow carrying capacity in the Project Area. The most significant agents include the local geology, regional seismicity, tidal processes, past subsidence in the region, a rising sea level, changes in land use in the watershed and on the floodplains, urbanization, channel improvement projects upstream of the Project Area, sediment accumulation from tidal and terrestrial sources, bridge constrictions and aggressive encroachment of in-channel vegetation along portions of the Guadalupe River. All of these agents and processes contribute to decreasing channel capacity with time and increasing annual maintenance requirements to meet present and future levels of flood conveyance required by the Santa Clara Valley Water District (District).

2.2. Study Purpose

The purpose of this study is to develop an understanding of the Lower Guadalupe River hydraulics with respect to its ability to convey flow and sediment materials now and in the future. The object is to gather the necessary historical and field information required to develop engineering (quantitative) tools, including a sediment transport model of the Lower Guadalupe River, to use to evaluate present and future sediment yield to the study reach, channel capacity, and maintenance requirements to meet planning, environmental and design needs of the Santa Clara Valley Water District. Both long term trends (on the order of decades) in river degradation and aggradation as well as episodic single flood events (days) are evaluated for both present (existing) conditions and for two alternative channel designs.



Note:
* The Corps of Engineer's Upper Guadalupe Study does not include Reach 6

base map provided by Santa Clara Valley Water District

Figure 2.1

Guadalupe River watershed map with project locations.

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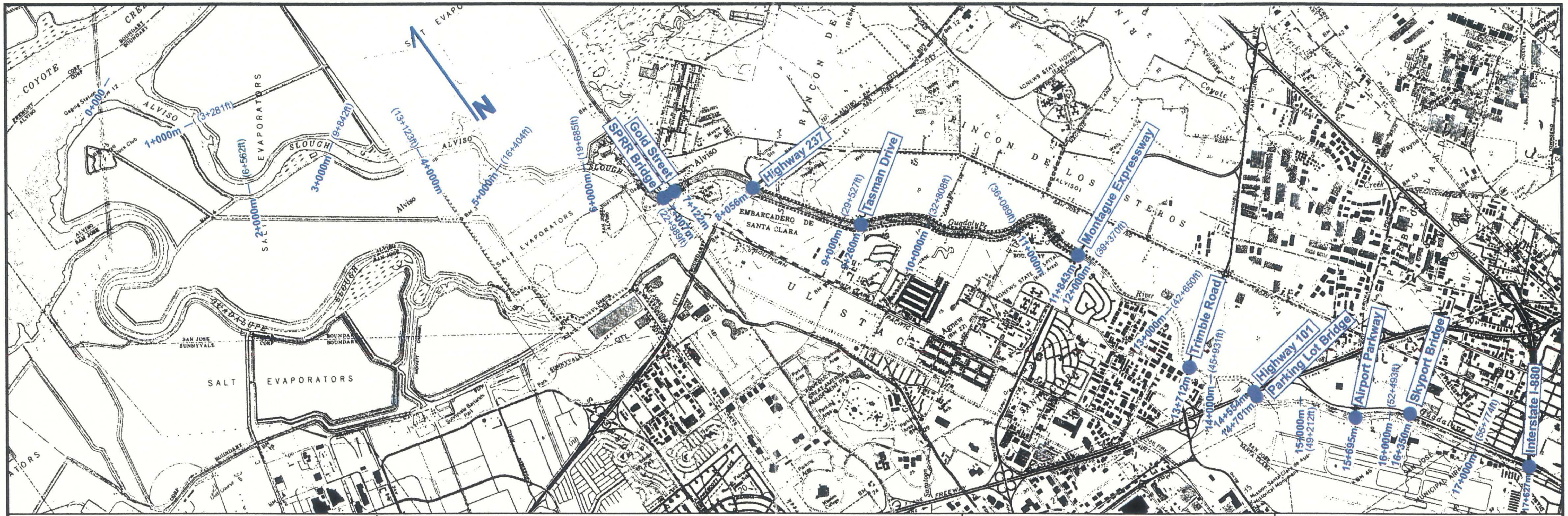

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notes:
- base mapping from USGS quadrangle sheets San Jose West, CA; Milpitas, CA; and Mountain View, CA.
- All stationing is in meters, with stationing in feet shown in parentheses using the conversion 1 m = 3.2808
- Stationing indicating bridge locations is approximate.

0 1 2
kilometers

Figure 2.2 Study area map of Lower Guadalupe River

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Information Sources

Chapter 3

3. Information Sources Utilized During the Study

Available information, reports and data were gathered and examined by project team members. Substantial information was provided by the Santa Clara Valley Water District regarding basin history, hydrology, channel geometry, existing hydraulic models, operations and maintenance, project planning and previous project design information. The District also provided available mapping and photo information as well as hydraulic data for the project area. Additional topographic, hydrologic and sediment data were obtained from the US Geological Survey (USGS), the Sacramento and San Francisco District Corps of Engineers, the Natural Resource Conservation Service, the City of San Jose and Santa Clara County. Tide information was obtained from the USGS and NOAA. Bridge details for the new California Department of Transportation (CalTrans) Airport Parkway bridge and preliminary mitigation site plans were provided by the District and CalTrans. Following is a summary of the sources of primary data used for the hydraulic and sedimentation analyses during the study.

- ***Basin History, Project Planning and Previous Project Design Information -*** This information was provided by the District.
- ***Basin Geology and Geomorphology -*** Data regarding the geology, regional seismicity and subsidence were obtained from the USGS, Department of Mines and Geology and from publications borrowed from State and University libraries in the area (refer to the References section at the end of the report).
- ***Model Bathymetry for the San Francisco Bay south of Dunbarton Bridge -*** This information was developed form NOAA navigation charts, NOAA hydrographic survey data, and recent aerial photo surveys processed by the USGS and Stanford University.
- ***Tide Data –*** Observed South Bay tide data were obtained from the USGS and Corps of Engineers.
- ***Guadalupe River Channel Geometry –*** The District provided several HEC-2 model geometry sets for the Guadalupe River from I-880 to the Bay. A listing of those data sets is provided in Section 7.2.2.
- ***Preliminary CalTrans Mitigation Planting Plans -*** Provided by the District.
- ***Bridge details and geometry –*** Provided by the District and CalTrans, primarily in the form of existing HEC-2 data files.
- ***Channel Roughness (Manning n) Values -*** The District provided a table (Table 7.2) of their estimates of "Composite n Values." NHC refined those values and converted them into separate values for overbanks and for the main channel in order to be compatible with HEC-6 and to more explicitly evaluate in-channel vegetation and roughness management alternatives.
- ***Sediment Transport Information –*** Historical sediment deposition volumes were derived from Corps of Engineers (COE, 1989) and District cross section information. Suspended sediment load information came from published USGS data collected at the St. John Street gage. Bed and bank material properties were developed from field samples collected and analyzed by NHC.
- ***Sediment Properties for Tidally Influenced Reach from Gold Street to the Bay –*** Sediment properties for the reach from Gold Street to the Bay were developed from core samples collected by NHC and analyzed by Professor Ray B. Krone (refer to Appendices B, C, and D).
- ***Hydrology and Flow Data –*** Hydrology and flow data for the Guadalupe River came from published USGS flow records for the San Jose Gage (Gage No. 11169000), from District provided hydrology and from the Corps of Engineers 1997 "Hydrologic Engineering Office Report" for the Guadalupe River (COE, 1977). Design hydrographs and peak discharges for specific frequency flood events were derived from the Corps (1977) Guadalupe River Hydrologic Engineering Office Report.
- ***Calibration Data –*** Peak flow and high water profile data for the 1995 Guadalupe River flood event were provided by the District for calibration.

Each of the following technical chapters expands on where important information came from and how it was utilized during the analyses. A complete list of "References Cited in this Report" is provided at the end of the Report.

Study Approach

Chapter 4

4. Study Approach: Developing a System Understanding

Evaluation of historical information, focused field investigations, and numerical modeling are used to quantify the complex hydrodynamic and sediment transport processes affecting channel capacity and hydraulics through the lower Guadalupe River downstream from Highway 880 to the SPRR Bridge at Alviso. Initial tasks develop an understanding of the key geomorphic, hydraulic and sediment transport characteristics of the study reach and provide an initial quantification of present (existing - baseline) channel conditions with respect to flood capacity and sedimentation processes. Sensitivity and comparative analyses are performed on several variations of a preferred project alternative being developed by the Santa Clara Valley Water District (District). Project alternatives are then refined into two preferred conceptual designs. Later tasks evaluate the hydraulic and sedimentation performance of the two preferred project alternatives for single event floods and long term average annual flow conditions. Following is a list of the most significant tasks performed during this investigation:

1. Gather and review previous studies, reports and available information, including historical maps, photos and data records;

2. Perform site evaluations and sediment sampling at selected locations in the study reach; develop sediment bed and bank material grain size information;

3. Perform a geomorphic assessment of the Project Area, develop an understanding of the primary agents, and physical processes affecting channel hydraulics and sediment transport along the Lower Guadalupe River, e.g., develop a "systems understanding of the Project Area;"

4. Assess historical information to develop model calibration data. Assemble available hydrologic and sedimentation data, channel maintenance and dredging records, tidal records, historical cross section and invert profile changes (survey data and photographs), changes in vegetation, and historical information on the construction and past performance of bridge crossings and flood control improvements throughout the study reach;

5. Estimate watershed sediment yield;

6. Assess existing (baseline) channel hydraulic conditions with the Corps of Engineers' (Corps) HEC-2 Program;

7. Utilize historical information to estimate future channel roughness and geometry conditions for the projected life of the project (assumed to be between 50 and 100 years from present);

8. Develop preliminary sediment transport trends and sediment loading for existing (baseline) conditions with the Corps' COE-SAM model;

9. Prepare and apply a two-dimensional hydrodynamic model to the lower end of the study reach to develop existing tidal boundary conditions;

10. Prepare and apply the Corps' HEC-6T for existing (baseline) conditions to evaluate sediment transport rates throughout the study reach for long-term and single storm event conditions;

11. Collect samples of Bay Mud in the tidally influenced zone, analyze those samples in a lab to determine their physical properties. Utilize these data and tidal forecast data with a long-term marsh accretion forecast procedure to estimate future tidal-channel invert profiles and downstream boundary conditions;

12. Perform preliminary assessment of proposed project components;

13. Evaluate performance of two preferred alternatives and compare their hydraulic and sediment transport performance to existing baseline conditions;

14. Attend project coordination meetings at the District and make presentations to their staff;

15. Prepare a report summarizing the hydraulic and sediment transport performance of the Lower Guadalupe River for existing conditions and for the two preferred project alternatives under average annual, design level single event conditions and for future long-term conditions.

Interim results were presented and discussed with District staff prior to proceeding with refinements to the proposed alternatives and final analysis of the preferred project alternatives. Two formal project coordination meetings and presentations, along with several interim meetings, were conducted during the course of the study to present and discuss project findings with District staff. The following project report provides discussions of the analyses, results and recommendations developed during the course of the "Sedimentation Investigation of the Lower Guadalupe River Project Area."

Historical Geomorphology

Chapter 5

5. Historical Geomorphology and Hydrospheric Processes

5.1. Physiography

Located in Santa Clara County, the Guadalupe River basin covers an area of approximately 160 square miles. The river originates in the Santa Cruz Mountains, located southwest of San Jose, and drains north through the heavily populated Santa Clara Valley and into San Francisco Bay (Figure 5.1). Major tributaries of the Guadalupe River are Alamitos Creek, Canoas Creek, and Los Gatos Creek. Basin relief is approximately 3,800 feet, with the highest point near Loma Prieta in the Santa Cruz Mountains and the lowest point near Alviso at San Francisco Bay.

Hydrology of the watershed has been greatly altered by human activity. The upper watershed is regulated by several reservoirs (Table 5.1) and the lower watershed is almost entirely urbanized.

Table 5.1
Reservoirs located in Guadalupe Drainage Basin

Reservoir Name	Basin Area Above reservoir (sq. mi.)	Year Built	Capacity (acre-ft)	Current Capacity (acre-ft)
Lexington	37	1952	20,200	19,830
Guadalupe	6	1935	3,700	3,230
Almaden	12	1935	1,800	1,550
Calero	8	1935	10,160	10,050

Combined, the four reservoirs cover a sub-basin area of 63 square miles, almost 40% of the watershed, and have a present storage capacity of 34,660 acre-ft. This capacity is used primarily for water supply, with some flood control benefits, although none of the reservoirs are operated for flood control purposes in the wet season. The non-regulated area of the watershed in Santa Clara Valley is largely urbanized and the Guadalupe River and its tributaries have been extensively channelized and leveed. The Guadalupe River reaches San Francisco Bay near Alviso where land use changes abruptly from densely urbanized municipal and residential areas to salt ponds and tidal marsh. The tidal influence of San Francisco Bay extends up the Guadalupe River to about the Montague Expressway. Tidal processes convey and redistribute sediments in the lower reaches of the river forming marginal marsh surfaces adjacent to the low flow channel within the leveed reach downstream of Tasman Drive.

5.2. Geology

5.2.1. General

The Santa Clara Valley is located in a structural depression of the Coast Range that has slowly infilled with sediments, creating a broad alluvial valley floor with deposits several tens of meters thick (Norris and Webb, 1990). The earliest of these deposits in the South San Francisco Bay area date from the early Pleistocene, about 1.5 million years ago (mya) (Atwater et al., 1977). Detailed discussion of deposits and sedimentary facies in the Santa Clara Valley is provided by Helley and Lajoie (1979).

Rocks in the upper Guadalupe River basin are highly sheared and faulted, typical of the geologically fractured and seismically active Coast Range. The San Andreas Fault borders the western edge of the basin and the Hayward and Calaveras Faults lie to the east. Numerous other unnamed faults also cross through the basin (DMG, 1992), all of them in a SE - NW orientation that follows the structural alignment of the Coast Range. Rock types consist primarily of sedimentary rocks of the Franciscan Complex, volcanics and metavolcanics, and ultramafic rocks whereas the lower basin is composed of Quaternary alluvium, some of it partially consolidated (Figure 5.2). The South San Francisco Bay and Guadalupe River estuary contains high concentrations of fine-grained bay muds though they do not achieve appreciable thicknesses until 4 to 5 miles downstream of Alviso (Goldman, 1969).

5.2.2. Sea Level Rise

Throughout the Holocene (last 10,000 years), the Guadalupe River has adjusted to consistently rising sea levels (Figure 5.3). Figure 5.3 shows that mean sea level has risen more than 40 meters during this period. Much of this rise occurred prior to about 7,000 B.P., coinciding with glacial retreat of the last ice age (Atwater et al., 1977). From about 6,000 B.P. to present, the trend line in Figure 5.3 shows a steady, gradual rise in sea level of 0.1 to 0.2 cm per year, indicating that it will continue to rise into the future. This suggests that sea level will be about 10 to 20 cm (0.4 to .8 ft) higher in 100 years, at the end of the design life of the lower Guadalupe River project. Increases in sea level will result in more extensive landward encroachment of tidal processes (water surface elevation, bay mud deposits and salinity) in the downstream end of the project area.

5.2.3. Land Subsidence

Santa Clara Valley has experienced regional land subsidence since at least 1906 (Figures 5.4 and 5.5). Much of this subsidence is attributed to large-scale groundwater overdraft that occurred prior to 1965, the year that state water deliveries began to arrive in the San Jose area. Subsidence and ground water table measurements from 1906 to 1995 reveal this trend (Figure 5.4). Figure 5.4 shows that almost all of the 14 feet of subsidence measured in downtown San Jose occurred from 1906 to 1965. Since then, the ground water table has reversed its downward trend and about 1 foot of additional subsidence has occurred.

Figure 5.5 shows that land subsidence along the Guadalupe River has not been uniform, at least for the period 1934 to 1967. Subsidence was at a maximum in San Jose at 8 feet (from 1934 to 1967) and lessened with distance outward, approaching 3 feet at the mouth of the Guadalupe River. Land subsidence at San Jose reached 12.7 feet in 1973 and has since been largely controlled by groundwater recharge. Land subsidence has altered the bed slope of the Guadalupe River throughout the affected area. Subsidence of the downstream-most portions of the study area has also contributed to recent dramatic land encroachment of tidal processes extending to about Montague Expressway.

5.3. Geomorphology

5.3.1. General

The lower Guadalupe River exists today as a managed floodway. The natural river channel has been straightened and confined by levees, giving it a relatively simple and consistent cross-sectional geometry that is narrower and deeper than the original stream condition. High flows are now confined to a narrow, steeper channel system with no opportunity to spread onto the floodplain. In addition to increasing flood conveyance capacity, these changes have reduced the potential for sediment accumulation and storage in the remaining floodplain and in-channel bars.

Geomorphically, the lower Guadalupe River can be divided into two distinct sub-reaches: tidal and non-tidal. The tidal reach extends from San Francisco Bay to Montague Expressway, the upstream tidal limit, and the non-tidal reach extends from Montague Expressway to the I-880 bridge, the upstream boundary of the study reach. A primary difference between the two reaches results from a change in elevation and slope at Montague Expressway. As the Guadalupe River enters the tidal zone, where its elevation is within the zone of influence of Bay tides, its slope abruptly shallows causing a decrease in sediment transport capacity that results in significant sediment deposition between Trimble Road and Montague Expressway. The tidal reach exhibits a continued reduction in channel slope with distance downstream and is characterized by tranquil low flow conditions with fine to very fine-grained bed and bank material. Typical cross-sections show relatively flat, narrow strips of tidal marsh bordering the low flow channel (referred to as marginal marsh features), particularly downstream of Tasman Drive, whereas upstream sections have more irregular topography. The non-tidal reach exhibits channel invert elevations above tidal influences and a steeper channel slope with higher energy conditions, reflected by the gravel bed material of the low flow channel, small gravel bars, and occasional steep, eroded banks such as those observed upstream of the Airport Parkway bridge.

Vegetation is also affected by tidal conditions. In fresh water areas upstream of Montague Expressway, riparian woodland consisting largely of willow, cottonwood, and box elder is dominant. Areas of fresh water marsh are found from Montague to Hetch-Hetchy, characterized by abundant bulrushes, areas of cattails along the channel bottom and decreasing riparian tree cover. Levee banks are covered by weeds and grasses and occasional brush. Salt marsh species become increasingly apparent downstream of Hetch - Hetchy, where cord grass and pickleweed compete with freshwater marsh species. Few, if any, trees are present in this area. Downstream of Tasman Drive, salt water species dominate and trees are absent.

5.3.2. Historical Geomorphology

This section reviews historical channel changes of the lower Guadalupe River system from 1857 to 1996. Levee construction, river training, dredging, and land subsidence due to ground water overdraft have all contributed to changes in channel morphology. Specific changes with respect to channel planform, invert profile, and channel geometry are examined in this section.

5.3.2.1. Changes in Channel Planform

An overlay map showing historical channel locations from 1857 to 1996 is shown in Figure 5.6. Historical data were obtained from a variety of sources and are summarized in Table 5.2. All map data were adjusted to fit a base map created from USGS 1:24,000 scale quad sheets. Road intersections and geographic landmarks were used to align channel planforms with the underlying base map. Maximum error of the planform information displayed in Figure 5.6 is estimated at +/-250 feet, though most areas are considerably less than this. This excludes the channel alignment from the 1800's which is considered to be less accurate (+/- 350 ft) due to inaccuracies in surveying techniques of the time.

Figure 5.6 illustrates how historical channel modifications have altered the natural river system through time. Prior to levee construction in 1963, the river channel was more sinuous and exhibited several meander bends, particularly in the area downstream of Tasman Drive. These meanders appear to have been fairly stable prior to levee construction which may be due in part to the composition of the channel banks. Observations along the channel course show them to consist of erosion resistant, fine-grained silt and clay at many locations. Private levees and river training may also have contributed to channel stability during this period.

Table 5.2
Mapping examined for Lower Guadalupe River study

Year Published	Year Planform Data Collected	Title	Source	Scale	Contour Interval	Comments
1870-1875	1857 – 1865	Rancho Ulistac, Rincon de los Esteros	U.S. Surveyor General	variable	N/A	Planform only, no contours
1953	1948	Milpitas, San Jose West	U.S.G.S. Quad Sheets	1:24,000	5 feet	no in-channel topography below water surface
N/A	pre-1963	Lower Guadalupe River Project, Alviso to I-880	Santa Clara Valley Water District	1"=300'	N/A	Planform only, no contours
1980	1979	Milpitas, San Jose West	U.S.G.S. Quad Sheets	1:24,000	5 feet	no in-channel topography below water surface
N/A	1983 – 1996	Lower Guadalupe River Project, Alviso to I-880	Santa Clara Valley Water District	1"=300'	N/A	Planform only, no contours

In 1963 levees were constructed, causing confinement and straightening of the river. Levee construction also eliminated the opportunity of high flows to spread onto adjacent floodplain, thereby increasing the effective unit discharge carried by the river for various frequency events. Sections of levee were realigned and/or raised between 0.3 m and 2.4 m in 1983, depending on location. In 1995, levees were raised again from the SPRR bridge to Highway 101 by 0.3 to 0.9 m as an emergency measure to provide added conveyance and freeboard. Both the 1979 and 1983-1996 channel alignments in Figure 5.6 show that the leveed flood control channel generally follows the trend of the historic alignment of the Guadalupe channel, though many of the smaller bends were straightened. Overall, levee construction both straightened and steepened the reach, reducing reach length from I-880 to the SPRR bridge by approximately 25%. Note that channel planforms in Figure 5.6 identifying periods 'pre-1963' and '1983-1996' represent channel planforms from interim periods between levee construction and channel modification.

Downstream of Alviso, channel patterns have changed dramatically from their original condition (Figure 5.7). At the turn of the century, the tidal zone downstream of Alviso was characterized by a distributary channel flow pattern, one in which the number of flow channels increase with distance downstream. Presently, all flows are contained within a single channel, Alviso Slough, flowing from Alviso to San Francisco Bay.

5.3.2.2. Changes in Channel Invert

Since significant levee modifications were made to increase flow capacity in 1983, the lower Guadalupe River has shown a general decline in elevation of the invert profile, most notably downstream of Trimble Road (Figure 5.8). Although some of this decline can be attributed to land subsidence (1 foot or less) during this period, a comparison of channel cross-sections from 1983, 1987 and 1996 (Appendix A) shows that the majority is the result of deepening of the low flow channel.

Because of significant land subsidence in the 1960's and prior, it is not possible to reliably correlate invert elevations from the original 1963 construction plans with 1983 and 1996 data. However, the influence of historic land subsidence on the river channel can be assessed by plotting a profile of land subsidence from Figure 5.5 along the channel invert (Figure 5.9). Figure 5.9 shows that historic land subsidence along the study reach varied from 0 to 8 feet from 1934 to 1967. Given that this period is roughly half that of total land subsidence (see Fig. 5.4), the total is probably closer to double this value. Because the center of maximum subsidence is located upstream of the I-880 bridge, the river downstream has experienced an overall reduction in channel slope whereas the reach upstream of I-880 has experienced an increase. In general, river reaches where slope is reduced tend to exhibit a trend toward decreased stream energy with less bed and bank erosion and greater potential for stream aggradation, whereas river reaches that are steepened tend to show an opposite trend. Identification of these trends on the Guadalupe River cannot be made however, because of artificial channel changes made during levee construction in 1963, immediately following the period of historical land subsidence.

5.3.2.3. Changes in Channel Cross-Section

A comprehensive data set comparing 1983/87 versus 1996 cross-section data is attached in Appendix A. Data from 1983 (Gold Street to Airport Pkwy), 1987 (Airport Pkwy to Highway 880), and 1996 were used for this assessment. The 1983 and 1987 cross-section data were obtained from the US Army Corps of Engineers (COE, 1989) and 1996 data were obtained from the Santa Clara Valley Water District. All data were converted to NAD88, where applicable, using the relation NAD88-NGVD29=0.81m. In general, the 1996 cross-section data in Appendix A exhibit a more well-defined low flow channel as well as aggradation along the channel banks when compared to 1983/87. This is particularly well illustrated in the tidally influenced zone downstream of Tasman Drive (Figure 5.10). In addition, almost all of the lateral bench (aggraded bank) deposits located downstream of Tasman Drive exhibit a nearly constant surface elevation from 2.25 and 2.75 m. This consistency of the lateral bench elevation is clearly related to tidal processes and defines an equilibrium elevation for sediment accumulation and the development of marginal marsh features within the leveed channel system.

The difference in cross-sectional area from 1983/87 to 1996 for all stations in Appendix A was calculated to identify sites of overall erosion or deposition. Area differences were extrapolated between cross-sections using the end-area method to calculate volumetric changes along the river. The results are shown in Figure 5.11 where the continuous line shows the cumulative volume difference (1983 minus 1996) with distance downstream from I-880. This line is negative for areas of net erosion and positive for areas of net deposition. The dashed line shows the net deposition per channel length. Thus, where this line trends down it indicates net erosion whereas an uptrend indicates net deposition. Examination of the cumulative volume difference trend line in Figure 5.11 shows overall erosion from 1987 to 1996 occurring upstream of Trimble Road. The volume difference trend line rises above zero at Trimble Road and continues to climb with distance downstream, indicating net deposition from Trimble Road to Gold Street. While the volume difference line indicates general trends, the deposition per channel length line reveals local variability in deposition and erosion for shorter reach lengths.

5.4. Sediment Sampling

Twenty (20) sediment samples along the project reach were collected at 19 sites for grain size analysis at both bed and bank locations. All samples were taken between 3 and 12 inches below the ground surface in order to minimize the impacts of surface features such as armoring or fine-grained lag deposits. The location and a description of each collected sample is given in Table 5.3. Grain size distributions for each sample are given in Appendix B.

Table 5.3
Sediment Sampling Results

Sample No.	station	LB or RB[A]	Comments[B]	Grain Size
1	marina	RB	collected at edge of distributary channel	Clay & silt
2	9+540	LB	collected on bench, 6 □ 12" deep	Clay & silt
3	10+200	RB	collected on bench between main and secondary channel, 6-12" deep, u/s of Hetch Hetchy	Silt & clay
4	12+540	LB	collected near levee toe, 6 - 12" deep, u/s of Montague	Silt
5	12+540	LB	collected near low flow channel, 6 - 12" deep, u/s of Montague	Silt
6	14+440	LB	collected 20' from waters edge, 4"- 6" beneath the surface	Gravel
6b	14+440	LB	collected 12" below the surface, same location as sample '6'	Gravel
7	13+410	RB	collected near side channel, 3" below surface, 0.25" of silt on surface, d/s of Trimble	Sand & gravel
8	12+540	RB	collected 50' from levee toe, 6 - 12" deep	Silt
9	12+900	RB	collected near transition from sand and gravel to primarily silt deposits, 6 - 12" deep	Silt & sand
10	13+170	RB	collected on gravel bar, 6 - 12" deep	Sand & gravel
11	13+170	RB	collected in sandy deposit, 6 - 12" deep	Sand
12	14+190	RB	collected 12" below the surface	Sand & gravel
13	14+880	RB	collected near low flow channel, 6 - 12" deep	Silt
14	15+100	RB	collected 12" below the surface	Silt - gravel
15	15+510	RB	collected 6-12" below the surface on gravel bar, in depositional area	Gravel
16	17+070	LB	collected on bench, 6-12" below the surface	Sand & gravel
17	16+680	LB	collected in area covered with veg., 6-12" beneath the surface	Silt - gravel
18	16+365	LB	collected 6-12" below the surface on gravel bar	Sand & gravel
19	16+010	LB	collected 6-12" below the surface on gravel bar	Sand & gravel

A. LB is left bank and RB is right bank.
B. Location and description of sediment samples collected on Lower Guadalupe River on 4/23/97.

Five additional sites were selected for core sampling, conducted to provide data necessary for predicting future rates of marsh aggradation within the tidally influenced reaches. Each site was selected based on its location in a relatively flat, undisturbed area of tidal marsh (Alviso Slough) on the Lower Guadalupe River. Sample sites are shown in Figure 5.12. Each site was sampled using a hand operated Shelby Tube Sampler mounted with a 3 inch diameter core pipe 30 inches in length. The sampler was driven completely into the soil but in some cases did not achieve full penetration of sediments into the core tube. This did not adversely affect prediction of marsh aggradation rates since full penetration was not necessary. Each core sample was tested at 2-cm intervals for moisture content, organic matter, and bulk density according to ASTM standard test methods. Details of the sampling and testing methods as well as test results are presented in Appendix C.

5.5. Current and Future Trends

5.5.1. Land subsidence, Sea Level Rise, and Seismicity

The processes of land subsidence, sea level rise, and deepening of the channel invert all contribute to an overall lowering of the Guadalupe River relative to sea level. This will result in upstream migration of the tidal zone as well as vertical accretion of the tidal marsh over time. Although land subsidence has reduced dramatically since the early 1960's, it has continued in San Jose at a rate of .05 feet per year since 1975 (see Fig. 5.4). Assuming this rate remains constant for the next 100 years, the design life of the project, then 5 feet of additional land subsidence could be expected to occur. In addition, assuming sea level will continue to rise at an approximate rate of .1 to .2 cm per year (see Fig. 5.3), then the 100-year total sea level increase will be on the order of 0.4 to 0.8 feet. Combined, land subsidence and sea level rise yield a cumulative drop in channel elevation relative to sea level of 5 to 6 feet (1.5 to 1.8 m) at San Jose in 100 years. Given that land subsidence has not been uniform historically (see Fig. 5.5), this decline is likely to be less with increasing distance downstream from San Jose. Thus, given the likelihood for continued subsidence and sea level rise, general aggradation and a reduction in channel capacity of tidally influenced areas of the Guadalupe River is anticipated. In addition, vegetation changes can also be expected to occur with upstream migration of the tidal zone.

The San Francisco Bay region is one of the most seismically active areas in North America. Thus, the potential for earthquakes to cause geologically based channel changes is high. Earthquakes can cause significant lateral and/or vertical coseismic adjustment of the ground surface. Liquefaction effects associated with ground shaking may also be produced in fine-grained tidal deposits or areas of loose, unconsolidated fill. This may cause localized settling of channel levees as well as redistribution of in-channel bed and bank sediments along the Guadalupe River.

5.5.2. Channel Cross-Section

As shown in Figure 5.11, the lower Guadalupe River is generally aggrading downstream of Trimble Road. Two significant morphologic features are almost always present in this reach. First, there is the development of a well defined low flow channel. Second, there is aggradation of the channel banks, forming bench deposits that border the low flow channel (see Fig. 5.10 and Appendix A). The tendency for the river to adopt this type of cross-section should be considered as part of any channel modification plan for this section of river. Channel alterations that diverge significantly from this morphology will require more upkeep and maintenance than do plans that incorporate the natural tendency of the river system.

Conversely, areas upstream of Trimble Road tend to show general erosion with time (see Fig. 5.11), due largely to a steeper channel profile. These areas also show a consistent trend toward development of a low flow channel, similar to areas downstream, but show greater erosion rather than aggradation of the channel banks. Perhaps some forms of biotechnical channel invert stabilization should be considered for this reach.

5.6. Conclusions

Conclusions from the geomorphic study are summarized as follows:

- The Guadalupe River basin has undergone tremendous land use changes since the turn of the century, including dam construction in the upper watershed, urbanization of Santa Clara Valley, and river training and levee construction along all major streams.

- Geology of the Guadalupe River basin is characterized largely by sedimentary rocks of the Franciscan Formation, volcanic and ultramafic rocks, sandstone, and alluvium. These rocks are deeply faulted and fractured and bordered to the west by the San Andreas Fault and to the east by the Hayward and Calaveras Faults.

- Historically active fault systems have a high likelihood of causing coseismic adjustment to the Guadalupe River basin within the design life of the project. Historically, coseismic adjustment is thought to have contributed to channel plan and profile adjustments elsewhere along the Guadalupe River.

- The lower Guadalupe River is affected by Holocene sea level rise which continues to occur at an estimated rate of 0.1 to 0.2 cm per year. Similarly, the Santa Clara Valley is affected by differential land subsidence, occurring at a rate of 0.05 feet per year in San Jose. If both sea level rise and land subsidence continue, then upstream migration of the tidal zone is expected to cause tidally induced aggradation.

- The tidal reach of the lower Guadalupe River (Alviso to Montague Expressway) is a depositional environment, characterized by low slope, low energy conditions with a well-defined low flow channel and aggraded channel banks. Bed and bank material consists primarily of silt and clay.

- The non-tidal reach (Montague Expressway to I-880) exhibits greater channel slope and higher stream energy, as well as both erosional (e.g. I-880 to Airport Parkway) and depositional zones (e.g. Trimble to Montague). Bed and bank material consists largely of sands and gravels.

- Levee construction in 1963 reduced channel length by 25%, mainly in the area downstream of Tasman Drive where several historic meanders were present. Levees also eliminated the former distributary flow pattern of the Guadalupe River downstream of Alviso and the ability of high flows to spread onto adjacent floodplains upstream of Alviso.

- A comparison of river profiles between 1982 and 1996 showed general declines in invert elevation of 0 to 2.5 m, primarily downstream of Trimble Road. This was mainly due to localized incision of the channel invert rather than general bed degradation.

- Volumetric comparisons made from 1983/87 and 1996 cross-section data show general erosion to have occurred from I-880 to Trimble Road, whereas deposition occurred from Trimble Road to Alviso. This deposition almost always occurred in the form of aggrading and widening channel banks which encroached the channel (see Fig. 5.10). The tendency for the river to adopt this type of cross-section should be considered as part of any channel modification plan for this section of river.

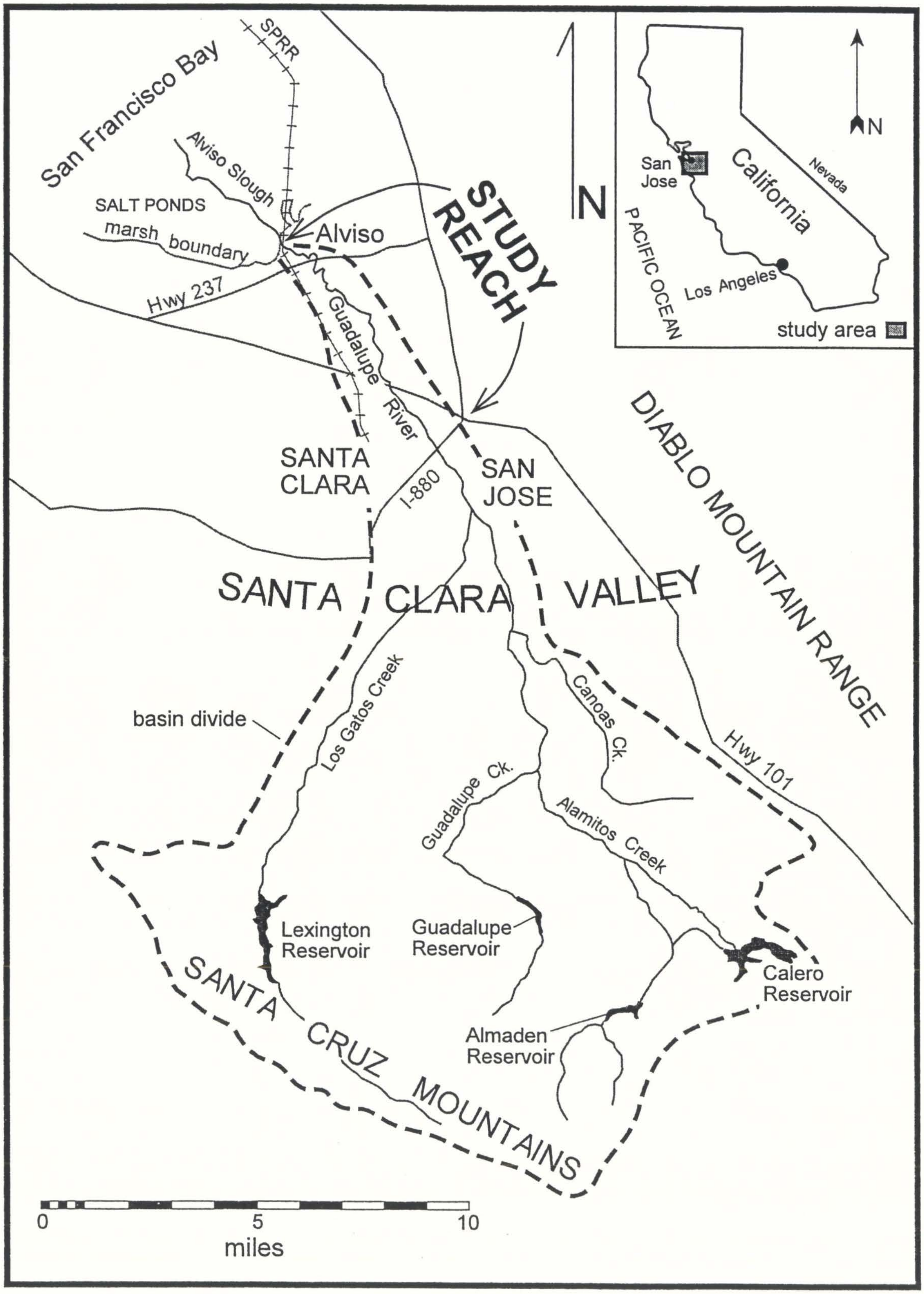


Figure 5.1 Study area map showing major features of Guadalupe River basin.

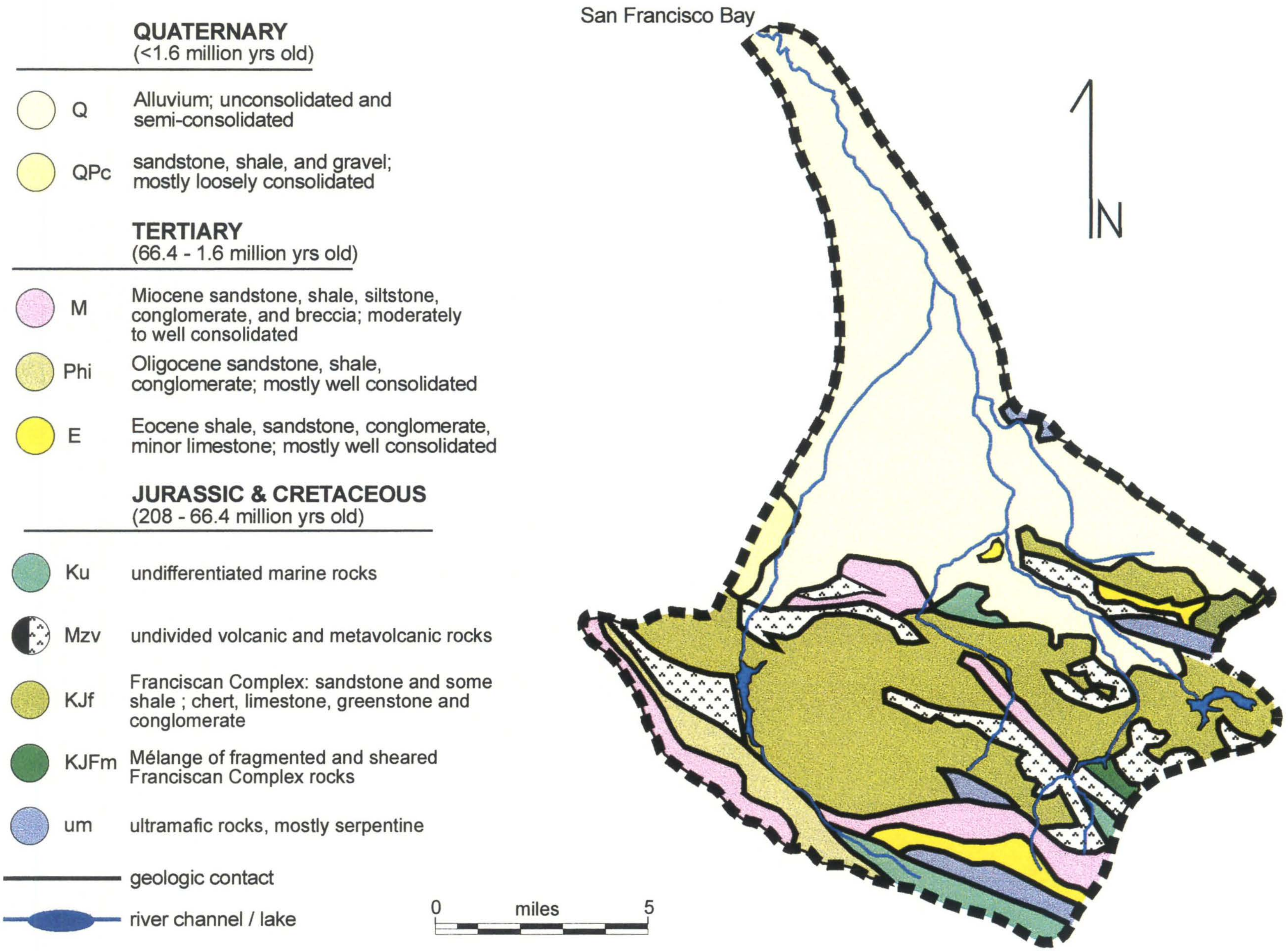

QUATERNARY
(<1.6 million yrs old)
Q
Alluvium; unconsolidated and semi-consolidated
QPc
sandstone, shale, and gravel; mostly loosely consolidated
TERTIARY
(66.4 - 1.6 million yrs old)
M
Miocene sandstone, shale, siltstone, conglomerate, and breccia; moderately to well consolidated
Phi
Oligocene sandstone, shale, conglomerate; mostly well consolidated
E
Eocene shale, sandstone, conglomerate, minor limestone; mostly well consolidated
JURASSIC & CRETACEOUS
(208 - 66.4 million yrs old)
Ku
undifferentiated marine rocks
Mzv
undivided volcanic and metavolcanic rocks
KJf
Franciscan Complex: sandstone and some shale ; chert, limestone, greenstone and conglomerate
KJFm
Mélange of fragmented and sheared Franciscan Complex rocks
um
ultramafic rocks, mostly serpentine
geologic contact
river channel / lake
0
miles
5
San Francisco Bay
N

Figure 5.2 Geologic map of Guadalupe River basin (from DMG, 1990). Faults are not shown.

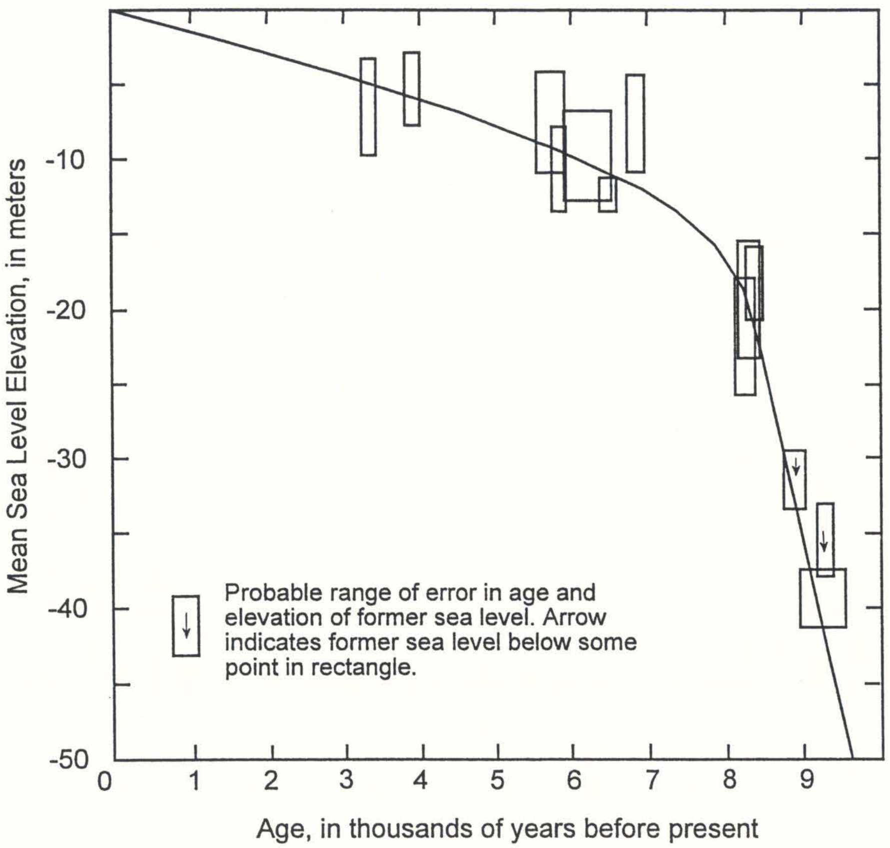


Figure 5.3 Holocene sea level changes in South San Francisco Bay (from Atwater et al., 1977). Sea level has risen approximately 50 meters through the Holocene (last 10,000 years), lessening to a rate between 0.1 and 0.2 cm/year during the last 6,000 years.

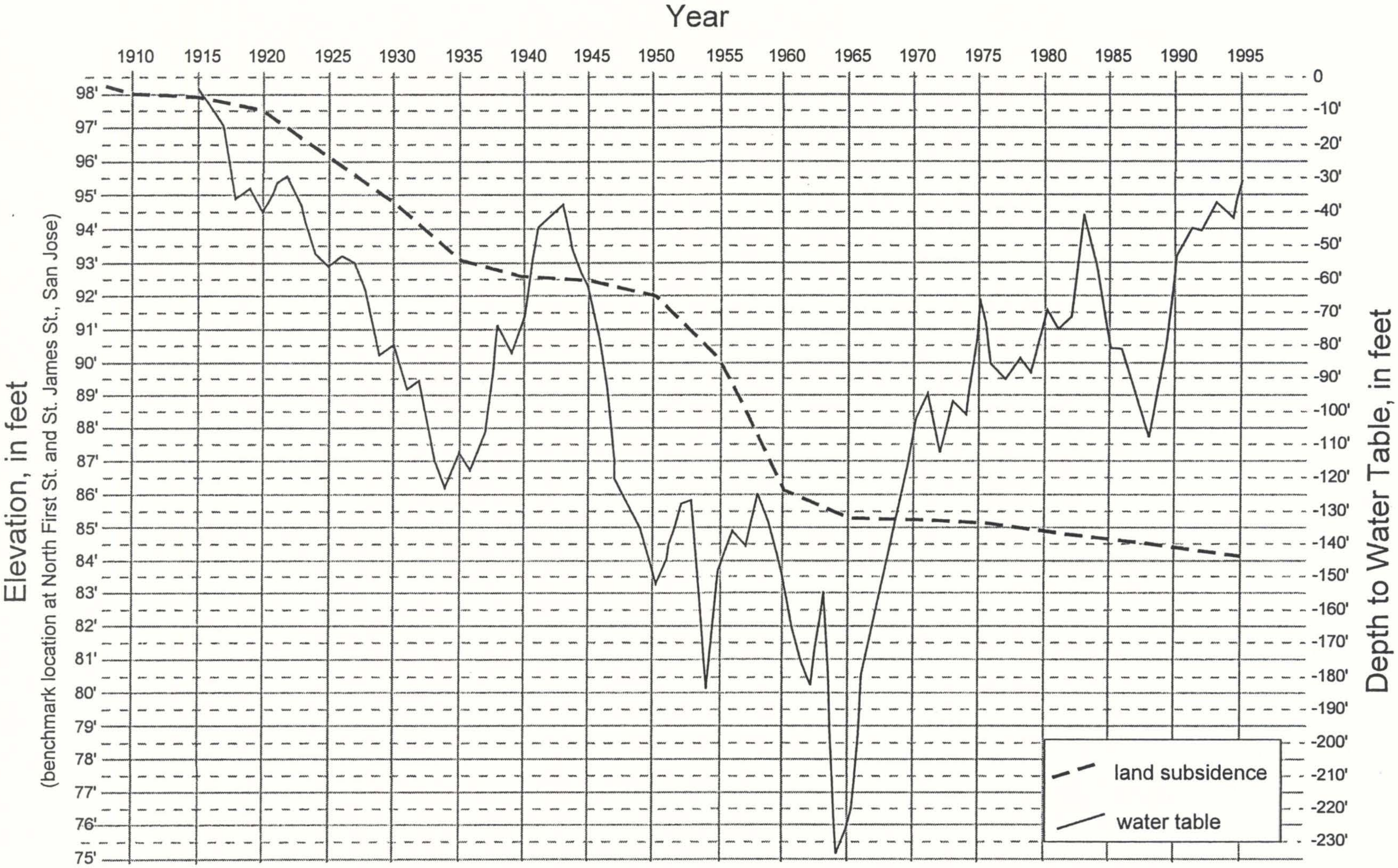


Figure 5.4 Land Subsidence and Depth to Water Table, San Jose, California (1906 - 1995) (from: SCVWD, 1995).

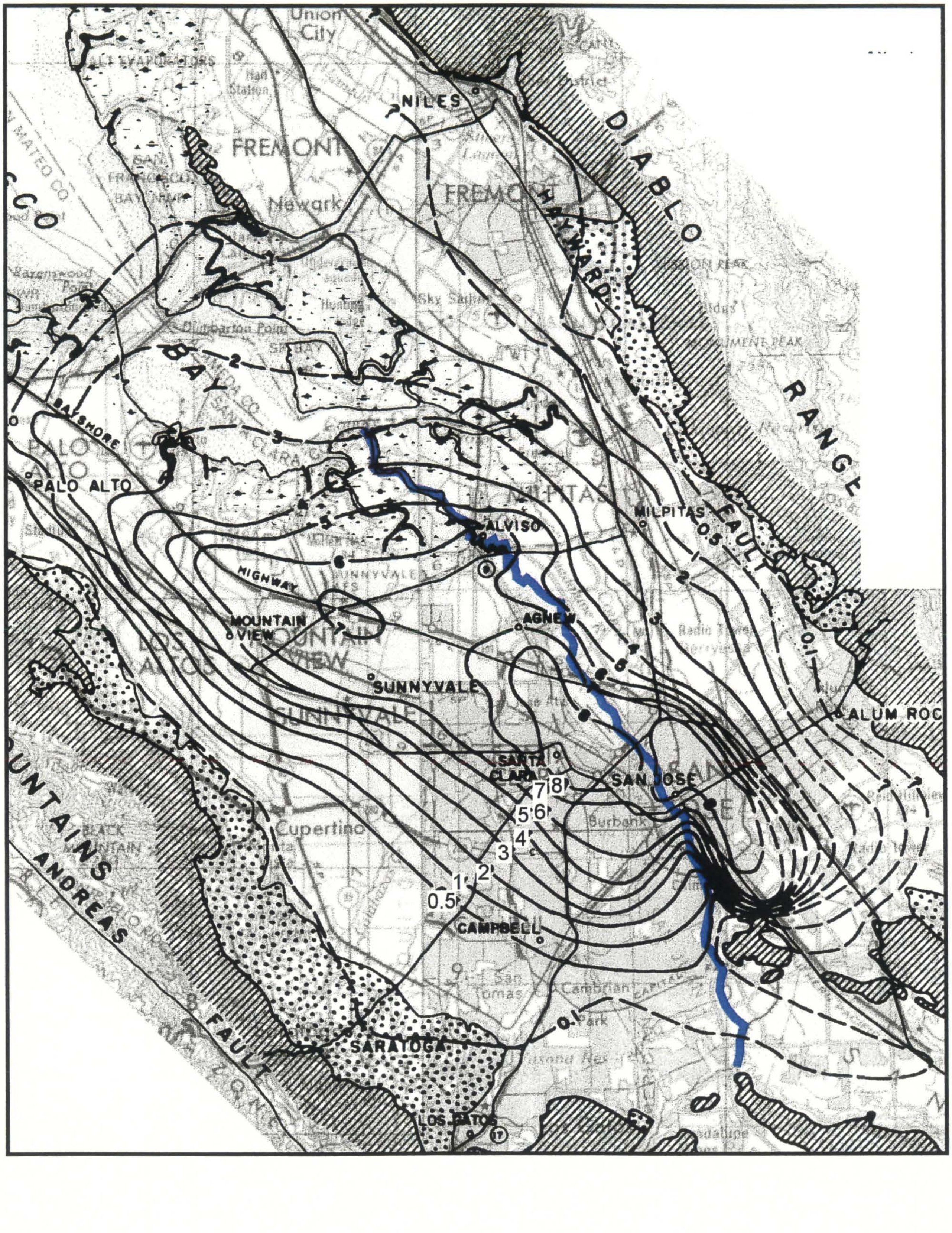
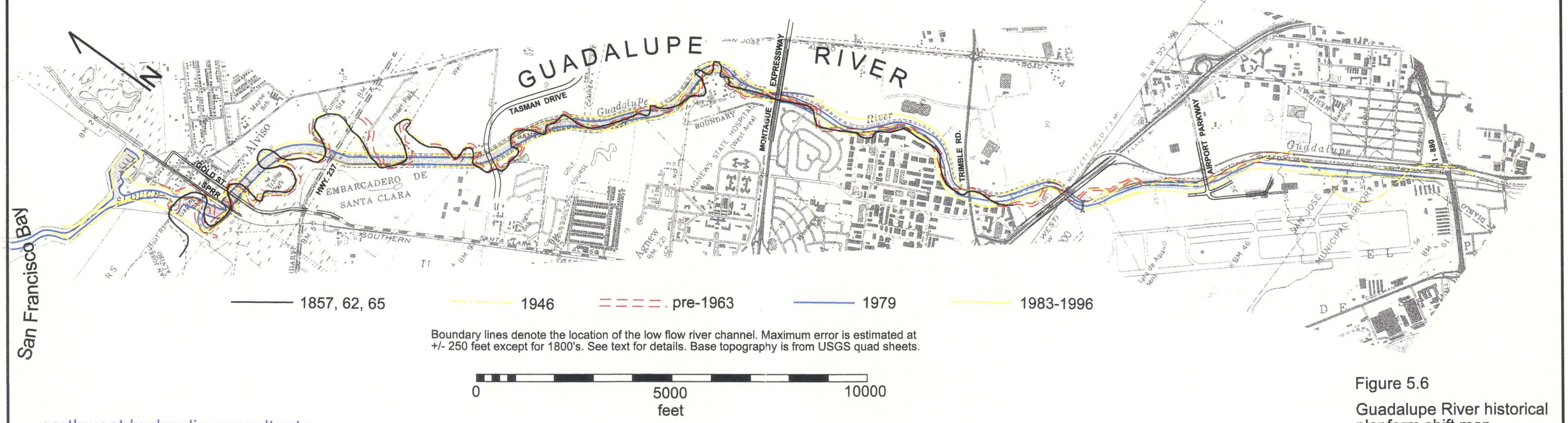


Figure 5.5 Map showing equal lines of land subsidence (in feet) in Santa Clara Valley from 1934 to 1967 (from Poland and Ireland, 1968) overlaid onto topographic mapping from the USGS. Guadalupe River is highlighted in blue.

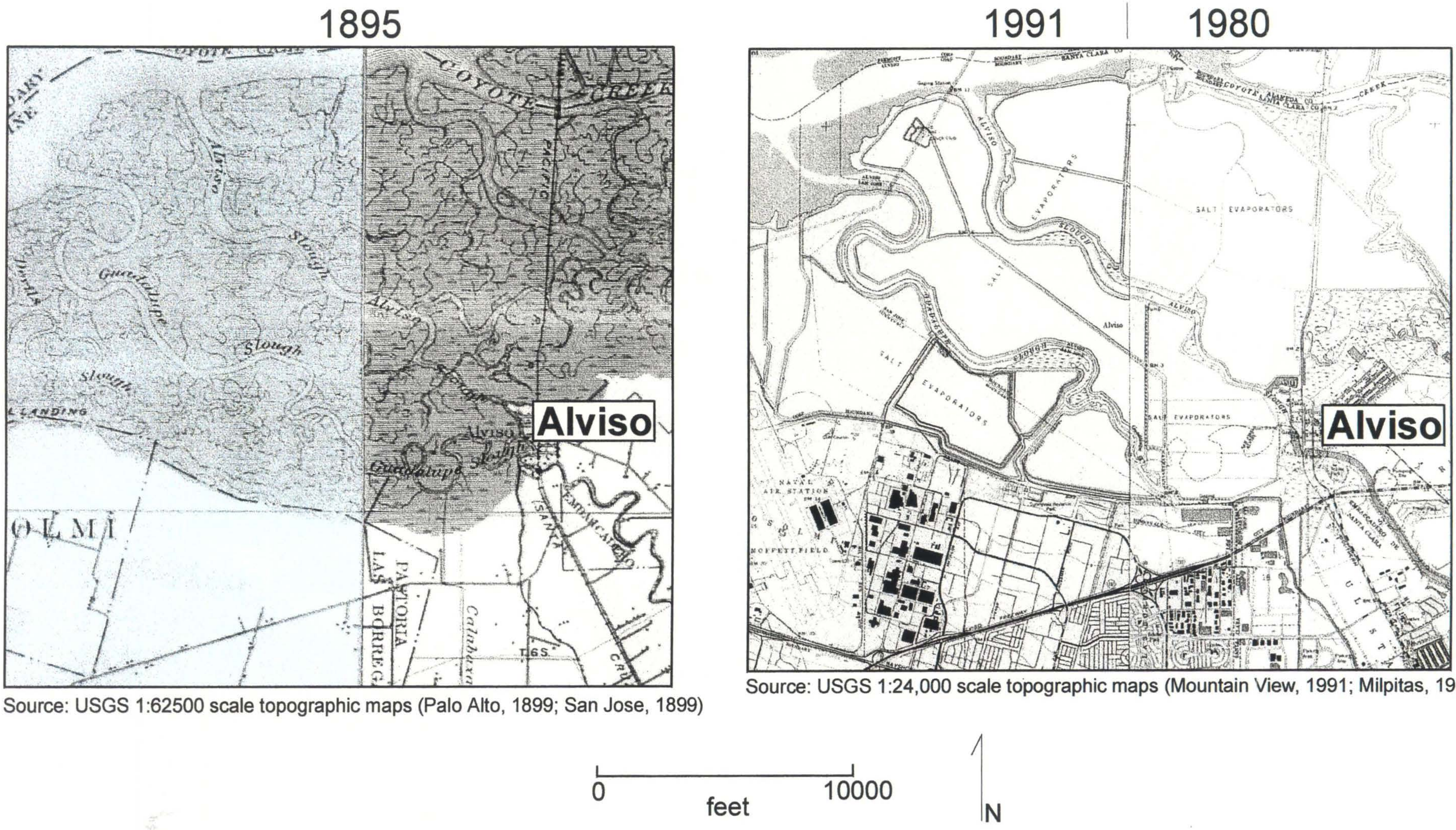


Boundary lines denote the location of the low flow river channel. Maximum error is estimated at +/- 250 feet except for 1800's. See text for details. Base topography is from USGS quad sheets.

northwest hydraulic consultants

Figure 5.6

Guadalupe River historical planform shift map.



Source: USGS 1:62500 scale topographic maps (Palo Alto, 1899; San Jose, 1899)

Source: USGS 1:24,000 scale topographic maps (Mountain View, 1991; Milpitas, 1980)

Figure 5.7 Changes in Guadalupe River planform in the tidal marsh donwstream of Alviso from 1895 to 1980, 91. The original condition exhibits many distributary flow channels whereas presently all flows are carried through Alviso slough.

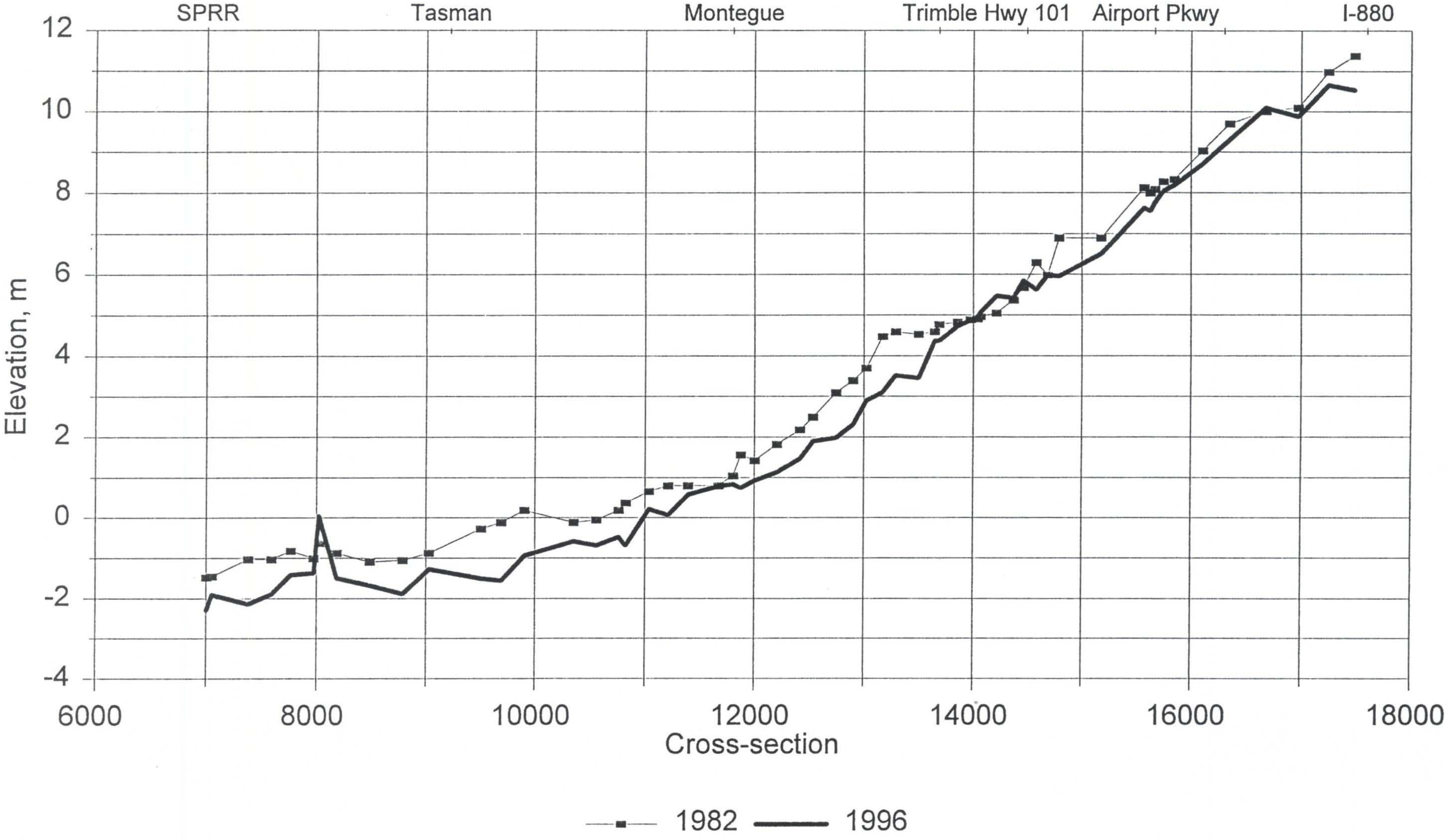


Figure 5.8 Guadalupe River invert profiles, SPRR to I-880 (from COE, 1989). All elevations set to NAD88.

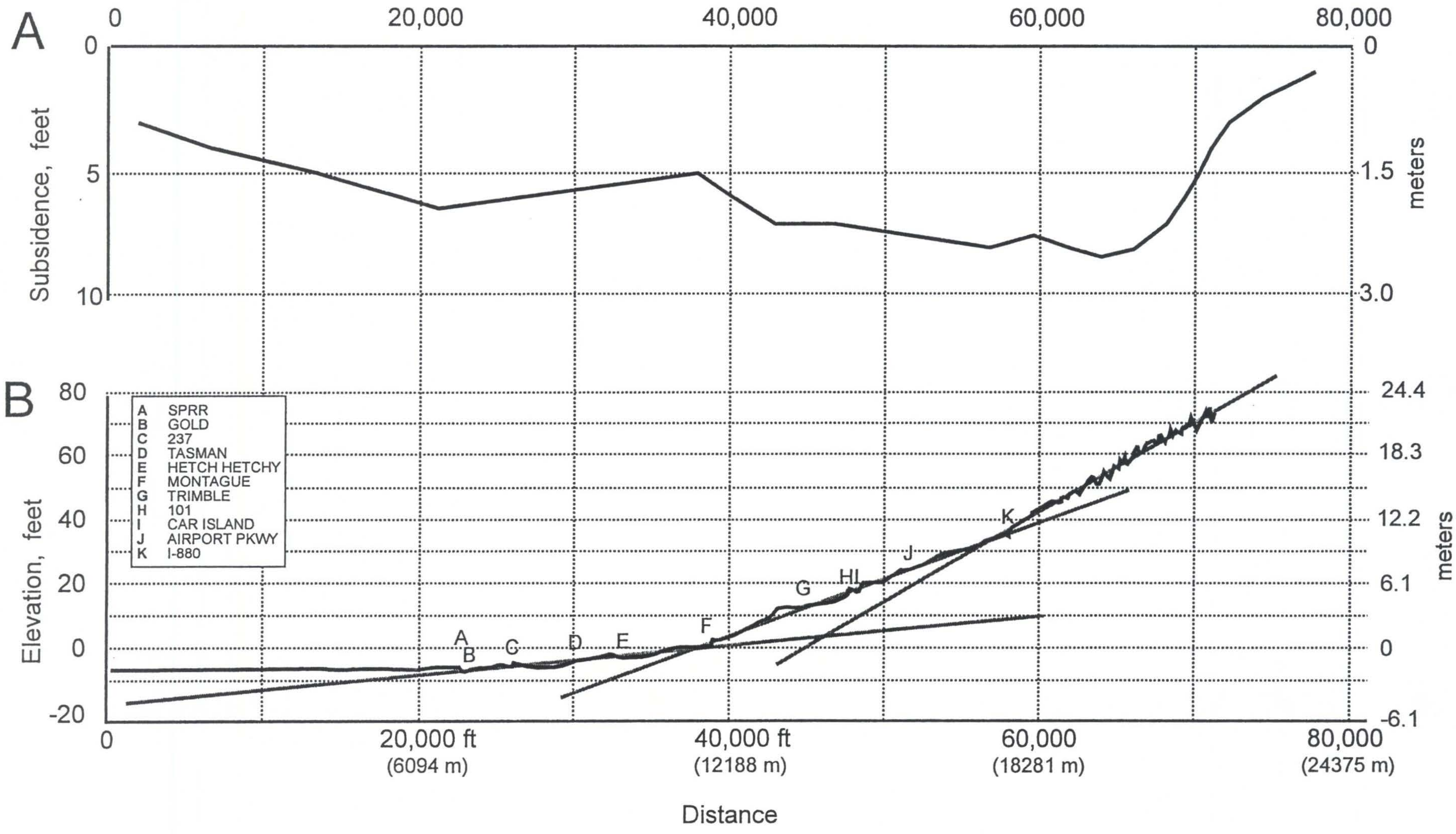
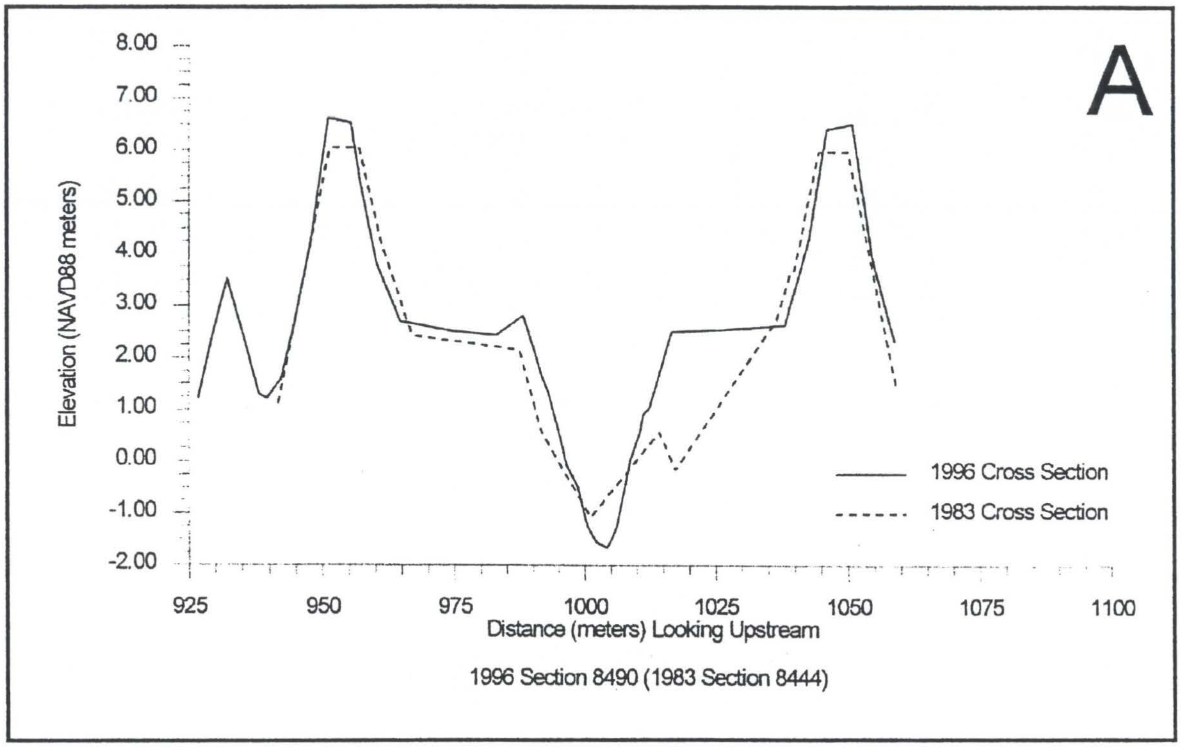


Figure 5.9 (A) Land subsidence along Guadalupe River from 1934 - 1967 (from Poland and Ireland, 1968) and (B) Guadalupe River profile (from COE, 1989). Lines identify regions of continuous channel slope. All data are set to NGVD 29.



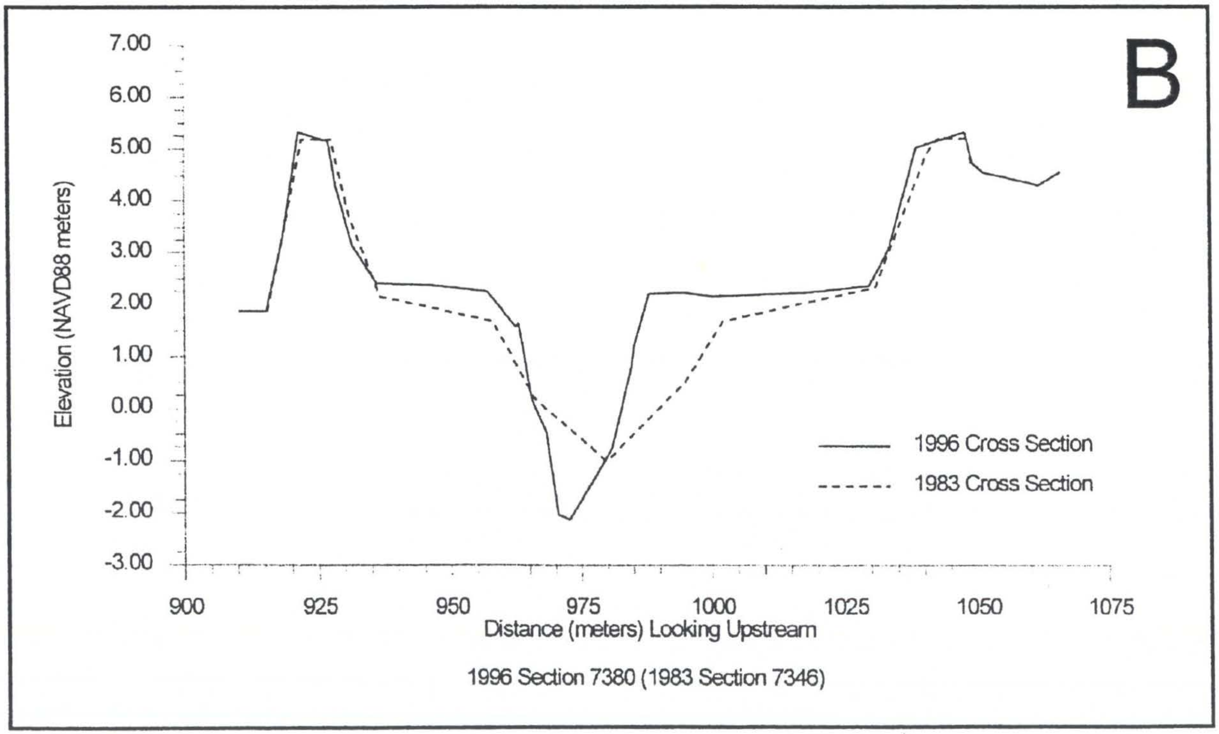


Figure 5.10 Cross-sections from 1983 and 1996 show changes in channel geometry at (A) located 460 m upstream of Hwy 237 and (B) located 270 m upstream of Gold Street bridge. The low flow channel exhibits varying degrees of incision and narrowing resulting from sediment accumulation between the leveed banks.

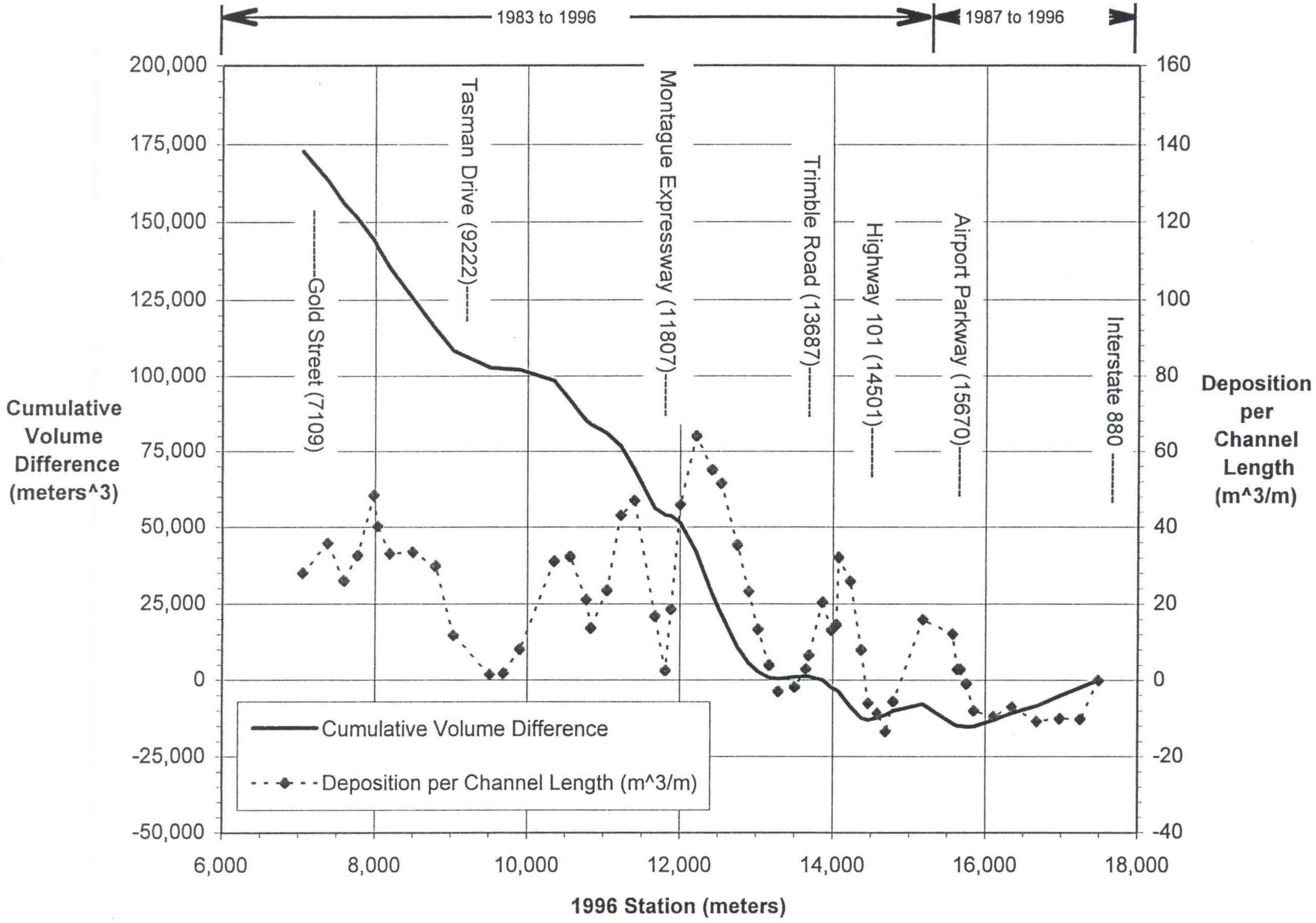


Figure 5.11 Cumulative Volume Difference between 1983/1987 and 1996 Cross Sections

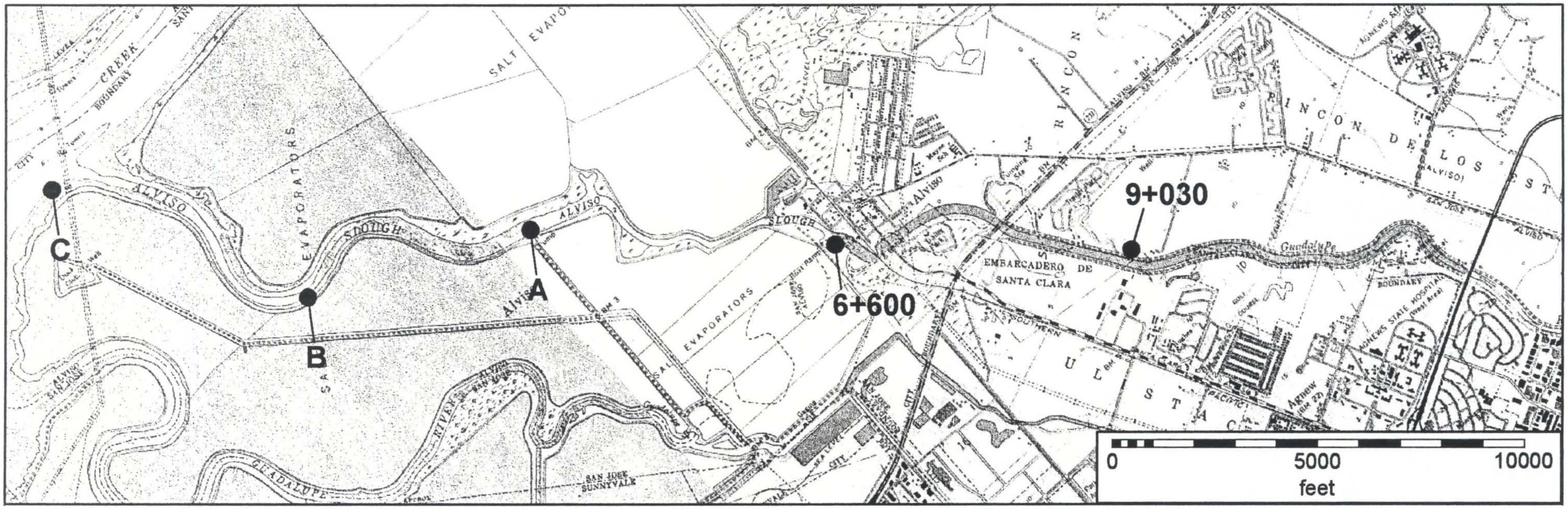


Figure 5.12 Location map of shelby tube core samples collected for Lower Guadalupe River sediment study. Cores A, B, and C were collected in February 1998 and cores 6+600 and 9+030 were collected in August, 1997.

Chapter 6

Tidal Processes and Boundary Conditions

6. Tidal Processes and Boundary Conditions

6.1. Introduction

Significant portions of the Lower Guadalupe River Project Area are influenced by tidal processes, including wind generated waves, vertical fluctuations in water surface, and the net accumulation of Bay muds. Channel reaches downstream from Montague Expressway and Tasman Drive begin to experience the landward extension of tidal influences due to the occurrence of mixed diurnal, asymmetric tides that occur in the South Bay. Twice each day (every 25 hours) two high and two low tides occur in the Bay. These tides generate landward flow of Bay water during rising tides (referred to as Flood Tide), and a Bayward excursion of flow during the falling tide (referred to as Ebb Tide). Near Alviso Slough at the downstream end of the Project Area, the typical vertical excursion of the water surface in the Bay ranges 6.8 ft. (2.07 m,mean range) and 8.5 ft. (2.59 m,diurnal range) from low to high tide. Therefore, the water surface elevation in the Lower Guadalupe River can be influenced by the magnitude and timing of high and low tides. This can affect flood stages in the lower-most regions of the river if the timing of a spring high tide occurs simultaneously with the peak river discharge during a runoff event.

Fine sediment materials suspended daily in the South Bay by wave action work their way back upstream into the Lower Guadalupe River with each flooding tide, resulting in a net long-term accumulation of Bay Muds in the bottom of the leveed channels. The rate and extent of this accumulation varies temporally and spatially within the system, but is observed to be significant in the South Bay. It is, therefore important to estimate the future magnitude and landward extent of the tidal accretion processes in order to properly evaluate flood control alternatives.

The following discussions describe the procedures used to develop the tidal boundary conditions used for this investigation.

6.2. Approach

Two-dimensional depth-averaged, time dependent hydrodynamic modeling was performed using the RMA-2 finite element model to assess the extent and degree of tidal effects on stage and flow in the lower reach of the Guadalupe River during low flow and storm flow conditions. In addition, the RMA-2 model was employed in a steady-state mode to develop lateral channel flow distributions for selected storm flows in the Montague Expressway to Trimble Road and I-880 to Coleman Avenue reaches.

RMA-2 is a finite element model for solution of the two dimensional depth averaged shallow water flow equations (King and Roig, 1988). The model is capable of simulating either time-dependant or steady state systems and is well suited to analysis of estuary or river systems. The RMA-2 program was developed under contract to the Corps' Waterways Experiment Station (WES), and serves as the basic hydrodynamic tool for the TABS-2 system developed by WES.

The model is capable of solving networks of combined one- and two-dimensional elements. The one-dimensional line elements are limited to channels having trapezoidal cross sections. However, the bottom width and channel side slopes may vary linearly along the element. RMA-2 permits control structures across both one and two-dimensional sections.

A "marsh element" or equivalent porosity formulation has been added to RMA-2 to improve performance when simulating areas of flooding and drying during tidal cycles or flood events. This method allows elements to become partially dry and permits the wetting and drying process to proceed in a stable manner. Application of the "marsh element" enhancement provides for a more explicit depiction of flow processes in partially submerged regions of vegetation and uneven ground as often is the case for the overbank areas of the Lower Guadalupe River and Alviso Slough reaches.

The following criteria establish baseline conditions for identifying tidal influences to the project:

- The finite element model representation in this project includes the southern portion of San Francisco Bay from Dumbarton Bridge and the Lower Guadalupe River (See Figure 6.1).
- Model bathymetry for the San Francisco Bay south of Dumbarton Bridge was developed from NOAA navigation charts, NOAA hydrographic surveys, and aerial photo surveys processed by USGS and Stanford University.
- The tidal boundary was set at the Dumbarton Bridge.
- The bridge hydraulics in the Lower Guadalupe finite element model were not explicitly modeled in the RMA-2 simulations. Rather, specific friction coefficients were manipulated to mimic the impacts induced by the structures (see discussions in *Section 6.4* for a detailed explanation of model calibration).

6.3. RMA-2 Model Representation

The finite element model representation used in the study (Figure 6.1) encompasses the San Francisco Bay south of the Dumbarton Bridge and the Lower Guadalupe River downstream of Interstate 880. A time-dependent tide stage was applied at the Dumbarton Bridge boundary using available tidal harmonic constants to calculate a tide. Guadalupe River flows were introduced at the I-880 upstream location as either constant or time dependent flows.

Model bathymetry for the San Francisco Bay south of the Dumbarton Bridge was developed from NOAA navigation charts, NOAA hydrographic survey data, and recent aerial photo surveys processed by the USGS and Stanford University. Bed elevations for the Guadalupe River were developed using the 1996 HEC-2 cross-section data and stream alignments provided by the Santa Clara Valley Water District. Detailed plots of the finite element network for the Lower Guadalupe River are shown in Figures 6.2 and 6.3. All bed and water surface elevations used in the two-dimensional modeling study are relative to NAVD-88.

The tidal boundary was set at the Dumbarton Bridge as there is available tide data for this site and the location is sufficiently distant from the Lower Guadalupe River so that proximity of the tidal boundary condition does not unduly influence stage at Alviso Slough during high flow periods. The Guadalupe River from Trimble Road to I-880 is represented with one-dimensional channel elements. The main focus of the two-dimensional, unsteady flow modeling was the tidal reach of the Guadalupe River. The two-dimensional network was extended upstream to Trimble Road as the channel flow distributions of the Montague to Trimble reach were of interest. A separate detailed finite element representation was developed to analyze lateral channel flow distributions for the I-880 to Coleman Avenue reach to be used as input for the sediment transport analyses.

During the March 1995 and February 1998 flood events, flood waters overtopped the Cargill salt pond levee across the slough from Alviso. Figure 6.4 identifies the location where flood waters overtop the west levee into the Cargill salt ponds. The figure also shows the detail provided in the RMA-2 finite element network to depict this internal boundary location. Levee overtopping in the RMA-2 simulations was simulated using a stage-discharge boundary condition located between sections 6+540 and 6+660. This coincides with a 122 m segment identified between sections 6+420 to 6+780 of the District's HEC-2 data as the low spot in the Cargill levee. HEC-2 data provided by the District shows levee overtopping at this section of levee as flow over a weir having a crest elevation of 3.38 to 3.48 m for a length of 122 meters. The weir crest expands in extent an additional 90 and 63 meters on each side up to an elevation of 3.81 m. This was approximated in the RMA-2 by simulating levee overtopping as flow over a broad crested weir of length 160 meters and crest elevation of 3.3 meters.

The hydraulics of the bridge structures in the Lower Guadalupe River were not explicitly modeled in the RMA-2 runs. The head loss for these structures was determined using the HEC-2 simulations for the range of flows. Side channel and bottom friction coefficients were adjusted for the finite elements representing these structures in the RMA-2 model representation until the change in water surface reasonably matched the HEC-2 values for the full range of flow conditions.

6.4. Model Calibration

Hydrodynamic model calibration is performed by varying bottom roughness coefficients and adding network refinements and modifications until adequate agreement is obtained between computed and observed parameters. Data available for calibration of the Lower Guadalupe River model representation consisted of the high water mark data provided by Santa Clara Valley Water District for the March 10, 1995 flood event. Observed South Bay tide data was used for calibrating and checking the model tidal hydrodynamics.

A water-level recording station was situated in Coyote Creek at Alviso Slough for the month of February 1979. This location is coincident with the approximate location of the downstream boundary of the HEC-2 model. Comparison of model simulation and observed tidal stage for this location provides a good measure of the RMA-2 representation of the South Bay system in providing an appropriate downstream tidal boundary for other analyses.

Figure 6.5 compares model results and observed tidal elevations in Coyote Creek at Alviso Slough for February 1-7, 1979. During this period the observed tide exhibits some baseline variability due to meteorological effects. Generally the model reproduces the observed tide very well, though overall the simulated tidal amplitude appears to slightly under predict the observed tide by less than 5%. Whether this is due to inaccurate bathymetry or a limitation of the tidal calculation used for the tidal boundary condition or the model dynamics is difficult to determine from available data. No observed data are readily available for the Dumbarton Bridge location during this period, therefore no evaluation of the computed tide could be made. However, Figure 6.5 demonstrates that the RMA-2 computed tide sufficiently reproduces observed conditions at the downstream tidal boundary in Alviso Slough.

The high-water mark measurements for the March 10, 1995 flood event provided data for calibrating model parameters for the Alviso Slough-Guadalupe River portion of the RMA-2 model representation. Manning's "n" values derived from initial HEC-2 runs of the Lower Guadalupe River provided the starting point in determining the friction coefficients to be used in the two-dimensional analysis. These values were then adjusted to improve the match between the observed and computed water surface. For Alviso Slough and the tidally influenced segment of the Lower Guadalupe River, channel areas below approximately +.80 m NAVD (mean tide level) were assigned a Manning's "n" value of 0.025. Channel areas between +.80 to 2.0 m (mean tide to mean higher high water) which are clear of plants or sparsely vegetated were assigned Manning's "n" values of 0.033-0.040. For those elements representing regions of marsh plants, a Manning's "n" value of 0.070 was used.

Calibration runs were first performed using steady-state conditions, that is a constant elevation at the Dumbarton Bridge boundary and a constant 11,000 cfs inflow at the upstream I-880 location. A dynamic run was later performed using estimated tidal conditions for the March 10, 1995 period and Guadalupe River discharge data from the downtown San Jose gage location, about 1.5 miles up river of I-880. For the March 10, 1995 date, no observed tide data were available for the Dumbarton Bridge location. However, the predicted vs. observed tide stage at the Golden Gate indicated a substantial storm surge for this date of +0.4 to +0.6 meters. For this reason, 0.4 meters was added to the predicted Dumbarton Bridge tide to produce a calibration tidal boundary condition.

Figure 6.6 presents the computed and observed maximum water surface profiles for the March 1995 flood. The computed water surface is generally within 0.1 and 0.2 m of the high water mark data. Some discrepancy occurs at sections 6+600 and 6+660 m, where the observed high water marks on the west bank are at elevation 2.83 m and 3.03 m respectively. These low values are difficult to explain in that the model does not reproduce the 0.85 meter difference between the east bank vs. west bank high water mark observations at section 6+600. There was no discernable difference between the steady-state and the dynamic results in the maximum computed water surface elevation above section 6+000. The reason for the similarity of the peak water surface profiles upstream of section 6+000 is discussed later.

6.5. Tidal Modeling Of Alviso Slough And Lower Guadalupe River

The calibrated RMA-2 model was applied to examine a range of Guadalupe River flows to determine the extent of tidal effects on flow and water surface elevation in Alviso Slough and further upstream, and to provide downstream boundary conditions for the HEC-2 and HEC-6T modeling. Tidal simulations were performed for Guadalupe River discharges of 2,300-, 6,700-, 11,000- and 17,000-cfs (2-year, 10-year, March 1995 flood, and 100-year events). The tidal boundary condition employed was the February 1-2, 1979 computed Dumbarton Bridge tide (Figure 6.5). This time period includes a maximum and minimum tide approximating the MHHW (+2.00 m NAVD) and MLLW (-0.56 m NAVD) water surface elevations for the Dumbarton Bridge location. This tidal amplitude represents approximately 2.6 meters (8.5 feet) difference in elevation between the MHHW and MLLW. Guadalupe River discharges were held constant for the duration of the simulation period to provide a meaningful comparison of tidal effects.

Figures 6.7 through 6.10 present computed water surface profiles in Alviso Slough and the Guadalupe River upstream to Tasman Blvd for the range of flood flows at time of minimum (February 1, 23:00) and maximum (February 2, 5:00) tidal elevation. For a discharge of 2,300 cfs, there is at least a 0.1 meter higher water surface elevation at the time of maximum tide up to section 7+900, upstream of the SPRR Bridge. For all other higher flows, there is almost no difference between the maximum tide and minimum tide water surface profiles upstream of the Southern Pacific Railroad Bridge located at section 7+002. As the water surface elevation decreases with low tide, cross-sectional flow area decreases and mean velocity and friction loss increases in Alviso Slough. At discharge levels of 6,700 cfs and above, these effects significantly increase the hydraulic gradient and negate any decrease in water surface elevation at the downstream end of Alviso Slough. The model results show that for a Guadalupe River discharge of 11,000 cfs, depth-averaged velocities at low tide are greater than 2 m/sec develop near the exit of Alviso Slough to Coyote Creek. This indicates a significant potential for scour when flood flow occurs at low tide.

Model results show only minimal flow over the Cargill levee for the 10-year (Q = 6,700 cfs) flood event. For the March 1995 flood event (Q = 11,000 cfs), the model estimates approximately 3,440 cfs flow over the levee at time of maximum tide, and 3,290 cfs at time of minimum tide. This represents flow out of Alviso Slough that is approximately 30 percent of the total peak flow reaching Alviso. Simulation results for the 100-year event (Q = 17,000 cfs) indicate a discharge over the levee of 7,300 cfs. Thus about 43 percent of the peak flow reaching Alviso is lost. No modification of the network or the stage-flow boundary condition was made to account for an increase in the length levee overtopping expected to occur for a flow of 17,000 cfs in Alviso Slough. Therefore, the present RMA-2 model configuration may underestimate flow over the levee for this discharge event. Accurate top of levee profile and channel cross-section data are necessary to refine these results.

In summarizing the results, the water surface profiles developed in the analysis indicate tidal effects on water surface elevation are small above Alviso Slough for Guadalupe River discharges of 6,700 cfs and above. In the calibration process, the March 1995 flood event was run in both a steady-state mode and with a dynamic tide and river discharges with no observable difference in the maximum water surface elevation above Alviso Slough. No simulations were performed for extreme high tide conditions in which a strong storm surge and spring tide coincide with a flood event. However, the dynamic and steady-state runs of the March 1995 flood suggest the water surface profile during an extreme tide condition can be analyzed with a steady-state model. Unsteady flow analysis has the potential to quantify the volume of flow loss over levees during flood events and different tides. Tidal flows (ebb and flood) can be greater than the stream discharges during low river flow conditions.

6.6. Two Dimensional Modeling Of Flow Split Areas On Guadalupe River

There are several locations from Coleman Avenue to Trimble Bloulevard where in-channel topographic or vegetative features result in flows becoming split around those features resulting in different amounts of flow moving parallel to each other in the river at different velocities and therefore, with different sediment carrying capacities. To assist with the evaluation of these areas in the project reach a separate RMA-2 model was developed with the ability to simulate the effects of channel obstructions. Results from these analyses were used to complement the mobile boundary analyses described in Chapters 9, 10 and 11.

6.6.1. I-880 To Coleman Avenue

A separate detailed RMA-2 model representation of the Guadalupe River between I-880 and Coleman Avenue was developed to assess the lateral flow distribution in this reach during high flows events. The 2-year (2,300 cfs), March 1995 (11,000 cfs), and 100-year (17,000 cfs) flood discharges were simulated using constant flow conditions. The downstream elevation boundary condition was specified to match HEC-2 derived water surface elevations at I-880 for the same discharges. These were 14.0, 16.0 and 17.0 m NAVD respectively for discharge rates of 2,300, 11,000 and 17,000 cfs. Manning's "n" friction coefficients used in the two-dimensional analysis were those contained in the NHC and Corps HEC-2 data sets. These were in general 0.06 for the channel and 0.09 for the left overbank area.

Model results in the form of velocity vector plots (referred to as flow field plots) for the reach are presented in Figures 6.11 through 6.13 for flows of 2,300, 11,000 and 17,000 cfs, respectively. The velocity vectors (arrows) represent the direction and magnitude of the flow by the orientation and length of each vector; longer vectors mean the velocity is greater. A scale vector is provided on each figure. Dry areas during low flow are shown as shaded zones with no vectors and, therefore no water flowing over that area. It is apparent from the computed results that significant flow leaves the main channel and flows parallel to it on the left in-channel bench areas recently constructed as parts of Contracts 1 and 2. The flows remaining in the main channel show higher velocities than those in the overbank areas.

Flow splits, the relative amount of flow remaining in the main Guadalupe River channel and the flow moving in the left overbank flow areas, are shown for selected sections in Figure 6.14. Figure 6.14, shows that approximately 60 to 75 percent of the total flow actually exists in the left overbank during higher flow conditions. These values for the approximate lateral flow distributions between the main channel and overbank areas were used to validate the new "looped channel - flow split" enhancements that were made to the HEC-6T code in order to better simulate the effects of the mid-channel berms and the complex lateral differences in sediment transport processes occurring upstream of Highway 880 during low and intermediate flows. This is discussed further in Chapter 8. This is a practical way of providing quasi-two-dimensional flow information to HEC-6T. Figure 6.11 clearly shows effects of the mid-channel berms in separating flows (at Q=2,300 cfs) from the main channel and where openings in the mid-channel berm allow flow to move between the flood plain and the channel depending on stage differences. It also shows how flows split around the existing island berm at the West Heading Street Bridge. Figure 6.14 shows how the relative percentage of the flow splits remain fairly constant for the range of flows investigated. This indicates that the effects of the berm separating the channel from the left overbank area limits communication between the two flow paths for the 2,300 cfs discharge event. It also indicates that during infrequent, high energy flood events mitigation plantings and other improvements located on the floodplain will be subjected to significant flow energies. This may affect long-term maintenance requirements on the floodplains.

6.6.2. Montague Expressway To Trimble Boulevard

A similar flow-split analysis was performed for the Guadalupe River between the Montague Expressway and Trimble Road. This reach has been widened with the installation of a maintenance channel area constructed to the east of the natural low flow channel. This analysis employed the same finite element network and approach used in the tidal dynamics investigation. However, the Montague to Trimble segment of the network is represented with extra element detail. Velocity vector plots for the 2,300 and 11,000 cfs discharge events are presented in Figures 6.15 and 6.16. Flow splits between the natural channel and the right maintenance channel are shown for selected sections in Figure 6.17. These data were evaluated and used to supplement the mobile boundary models discussed in Chapters 9, 10 and 11. The figures clearly indicate the greater flow capacity of the maintenance channel. The two dimensional results also show that just downstream of Trimble Road significant flow crosses over from the left bank to the higher channel capacity area on the right bank. This type of two-dimensional analysis combined with field monitoring during and after high flow events can significantly aid in the design of sediment management areas along the river.

The following section discusses tidal accretion processes and summarizes results from field work and computer forecasts of future ultimate marsh profiles in the Lower Guadalupe River.

6.7. Analysis of Marsh Accretion

The South Bay is a very shallow bay having predominantly a mud bottom. During low tide, large mud flats are exposed. Daily breezes during the spring and summer create waves which suspend the sediment, and the suspended clays and silts are transported with tidal circulation into the marsh areas and mud flats during the higher tide levels. The slow currents in the sloughs and mud flats facilitate deposition of the suspended particulates.

Tidal marshes that are built by accretion (accumulation) are typified by a horizontal planar surface with a dendritic, serpentine channel structure. These types of meandering slough patterns are typical in naturally occurring tidal marsh areas. The temporal history of marsh deposits can be represented by the accumulation of sediment at a single location on the plane. A simple mass balance of suspended particles per unit area of bed at a specific location was used to simulate the site's accretion over a long period of time. Utilizing a computer allows for expedient computations in a short amount of time which represent very long durations of accumulation or accretion.

The mass balance equation used in this study[1] was:

$$(Y_w - Y_m)\frac{dC}{dt} = -w_s C + (C_* - C)\frac{dY_w}{dt}$$ **Equation 6-1**

and

$C_* = C_0$ for $\frac{Y_w}{dt} > 0$
$C_* = C$ otherwise

where

C = the suspended sediment concentration at the site;
C_0 = the suspended sediment concentration in the flooding water;
Y_w = the elevation of the water surface;
Y_m = the elevation of the marsh surface;
t = time;
w_s = the settling velocity of the suspended sediment.

The median settling velocity can be represented by:

$$w_s = AC^{4/3}$$

where A is an empirical coefficient (110 for C in gm/cm^3 and w_s in cm/s). The rise of the bed elevation "ΔY_m"due to accumulation of depositing sediment during an integration timestep "Δt" is:

$$\Delta Y_m = \frac{AC^{7/3}\Delta t}{C_s}$$ **Equation 6-2**

where C_s is the concentration of mineral solids in the marsh soil. The marsh forecast model required historic tides, the initial bed elevation, bed subsidence, the dry density of the sediment deposit, and the marsh elevation at a known time (for calibration purposes).

Records of tides at the San Francisco Presidio, California, are the longest tidal records in the United States. A review of the tidal records for the San Francisco Bay indicate a trend of rising sea level for the area on the order of approximately twenty (20) centimeters per century (or, 0.20 cm/year). Figure 6.18 depicts annual MSL data from the Presidio Gage. A linear fit to the 1929 to 1991 data is used to extrapolate the mean sea level into the future.

Groundwater pumping from numerous wells throughout the Santa Clara Valley dewatered the aquafirs and induced subsidence of the ground surface throughout the San Jose area. The maximum subsidence was approximately 12.7 feet in downtown San Jose. In 1973, an aggressive program of groundwater recharge was initiated, and the results have slowed or diminished the subsidence in some areas. Subsidence from 1934 through 1967 is depicted in Figure 5.5 located in Chapter 5. Annual subsidence based on trends depicted by Figure 5.4 (see Chapter 5) were quantified, and subtracted from the calculated rise in bed surface elevation due to tidal accretion.

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[1] R.B. Krone & Associates, "*Calculation of Effective Suspended Solids Concentrations in Alviso Slough, California.*", Davis California, 1998.

The sediment concentration in the in-flowing water is adjusted until the model reproduces the historical measured surface elevation. Once calibrated to historical data, the timeframe is extended out into the future to predict an estimated future marsh surface elevation. Four sites were used in the simulation to derive future accretion elevations for the Guadalupe River and Alviso Slough riverine system. (The sites are depicted in Figure 5.12 in Chapter 5 as sites A, B, C and 6+600.) River stationing (in metric units) for sites A, B, and C correspond to 4+050, 2+100, and 0+000, respectively.

Core samples for laboratory analysis were collected at the sites by NHC, and the analysis was performed by Sunland Analytical Lab, Inc. The moisture contents, bulk densities, and organic matter content (by ashing) were determined from slices of the cores. Results of the lab analysis are published in the R.B. Krone & Associates Report (Krone, 1998), and are included in Appendix D.

Calculations of sediment accretion at the three downstream stations (0+00, 2+100, and 4+050) began at year 1850, and at year 1963 for the upstream most point (Station 6+600). The upstream point's starting elevation corresponds to the dredging depth as a result of maintenance activities completed in 1963. The results of the analysis are presented in Table 6.1 below, as well as in Figures 6.19 and 6.20.

Table 6.1
Projected Marsh Soil Surface Elevations*

Station (meters from Bay)	Suspended Sediment Concentration (mg/l)	1998 Elevation (m)	Year 2025		Year 2050	
			Elev. (m)	Elev. > MHW (m)	Elev. (m)	Elev. > MHW (m)
0+000	262	2.29	2.51	0.41	2.64	0.48
2+100	209	2.19	2.43	0.34	2.57	0.41
4+050	242	2.20	2.44	0.34	2.58	0.41
6+600	243	2.08	2.34	0.24	2.53	0.37

*1988 NAVD datum.

If no tidal prism increases are created, such as connection(s) to nearby (interior) salt ponds, the tidal flows in the slough channel should increase only slightly with increased landward intrusion of the tides with sea level rise. The marsh-channel cross-section configuration should remain. Both channel invert and marsh elevations should rise together.

These projections show approximately a 1 to 1.5 foot (0.3 to 0.46m) rise in the Guadalupe River marsh soils due to the combination of sea level rise and tidal accretion processes. These projections are based on the assumption that regional subsidence has ceased and the rate of sea level rise continues at a rate of approximately 0.20 cm/year out to the year 2050. A greater rate of rise in sea level or re-enactment of subsidence will result in a faster rate of accretion and a greater difference in invert elevations than projected.

6.8. Summary of Effects of Tidal Accretion Processes In Project Area

The tidal reaches at the outlets of Guadalupe River are diked sloughs that cross former marshland. Prior to the construction of the levees, slough channel cross-sections were maintained by flows required for flooding and draining the large adjacent marsh lands, and to a lesser (and more occasional) extent by winter flows from Guadalupe River. The levees greatly reduced tidal flows in the sloughs, and suspended sediment in the flooding and ebbing bay waters rapidly accumulated to form marginal marshland and a much smaller tidal channel between the levees.

The sediments that deposit between the levees are comprised of material that is suspended in South Bay and South San Francisco Bay and its tidal margins by waves created by daily onshore spring and summer winds and winds during winter storms. Typical daily suspended sediment concentrations in the South Bay are approximately 200 to 1,500 mg/l. This sediment is soil eroded from the terrestrial lands that has entered the San Francisco Bay system with winter runoff from contiguous streams, including the Guadalupe River. Most of the material comes from the Central Valley, where it initially deposits in the upper bays. Subsequently, it is repeatedly resuspended by waves as it is circulated throughout the system by tidal currents. This complex long term mixing process winnows and circulates finer particles, with the result that the material found in South Bay and its sloughs is mud comprised of very fine clay materials.

While an initial deposit of Bay mud in a slough channel is soft, it develops significant resistance to erosion by daily currents within a few days. The material forms a gel whose strength can only be reduced by deformation. Waves cause repeated deformation of the bed, and the material is liquefied and easily resuspended. Materials that remain on the bed in the active tidal channels become relatively stable with time and resist significant deformation from normal daily tidal hydraulics, including tidal currents and diurnal wave action. Wave action is negligible in the sheltered waters between levees, however, and is not a significant process for eroding the bed there. Material deposited on the marginal marsh surfaces within the leveed channels is stabilized by vegetation over time and by the partial drying that occurs when it is exposed between times of high tides. Sediment will continue to accumulate on the marginal marshes along the sloughs as sea level rises, and the slough channel will retain only the cross-section that can be maintained by daily tidal flows for flooding and draining the marginal marsh and for the tidal prism of the channel itself. Therefore, channel cross sections in the leveed sloughs will continue to build marginal marshes with narrow, deep channels with invert profiles and cross sectional capacity to carry daily tidal flows. Based on results presented in the preceding section, the projected magnitude of tidal accretion in the Lower Guadalupe River is on the order of 1 to 1.5 feet (0.3 to 0.46m) by the year 2050. These projections are based on the assumption that regional subsidence has ceased and the rate of sea level rise continues at a rate of approximately 0.20 cm/year out to the year 2050. A greater rate of rise in sea level or re-enactment of subsidence will result in a faster rate of accretion and a greater difference in invert elevations than projected. Obviously, a rise in channel invert and tide elevations between the levees leads to reduced upstream channel capacity and increasing flood risk with time.

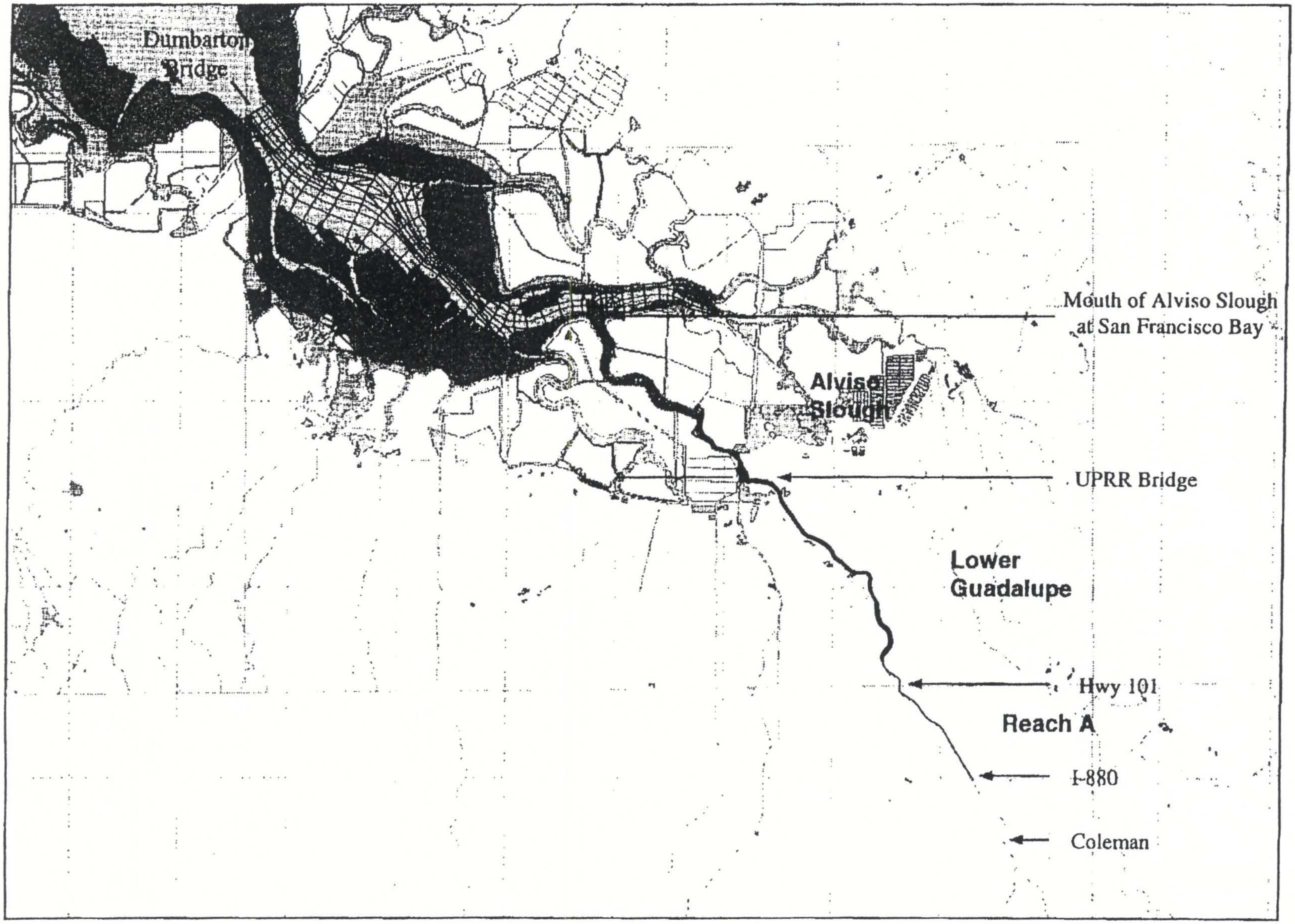


Figure 6.1. Finite Element Representation of the Lower Guadalupe River and South San Francisco Bay, Dumbarton Bridge to I-880.

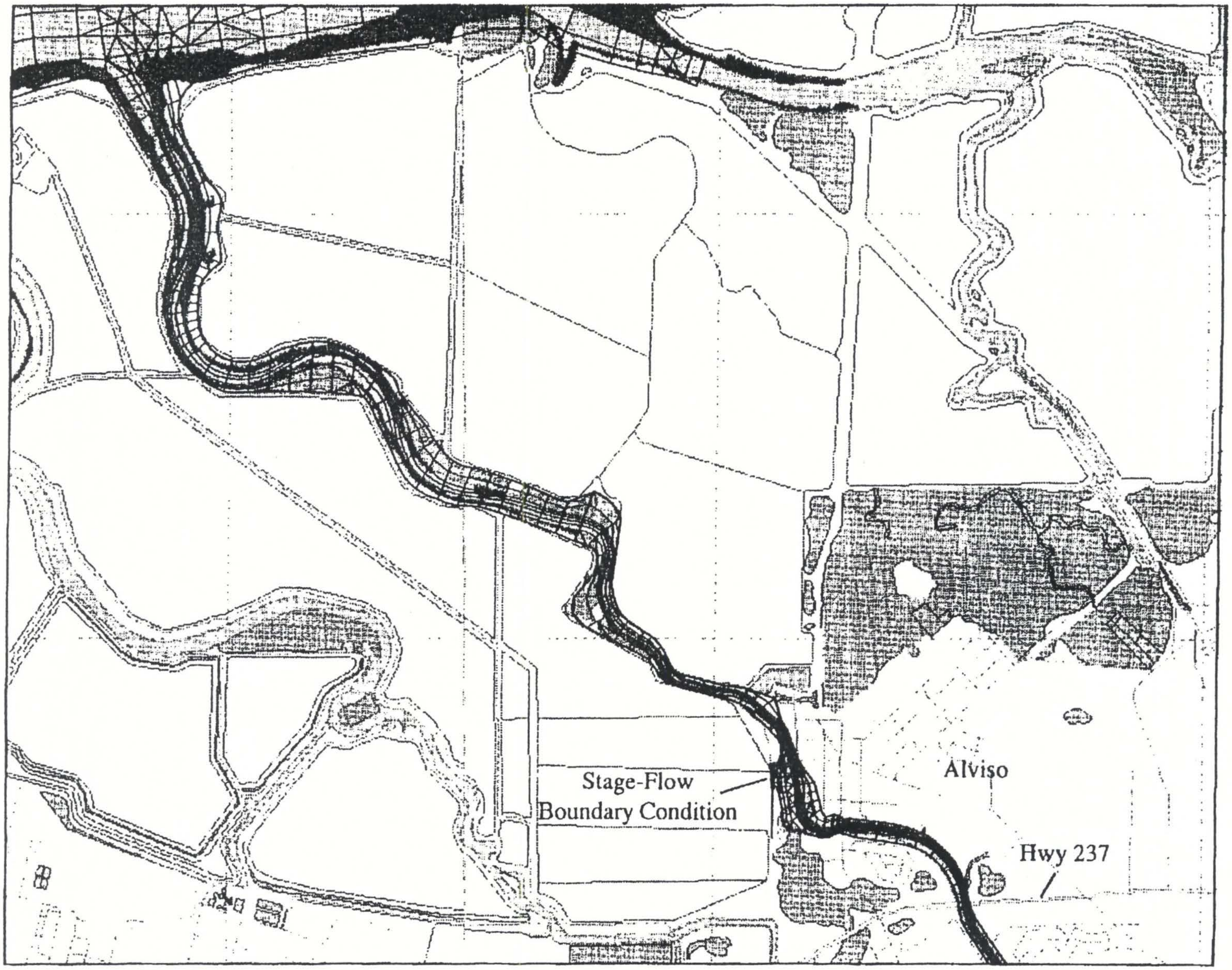


Figure 6.2. Detail of finite element network for Lower Guadalupe River, downstream of Highway 237.

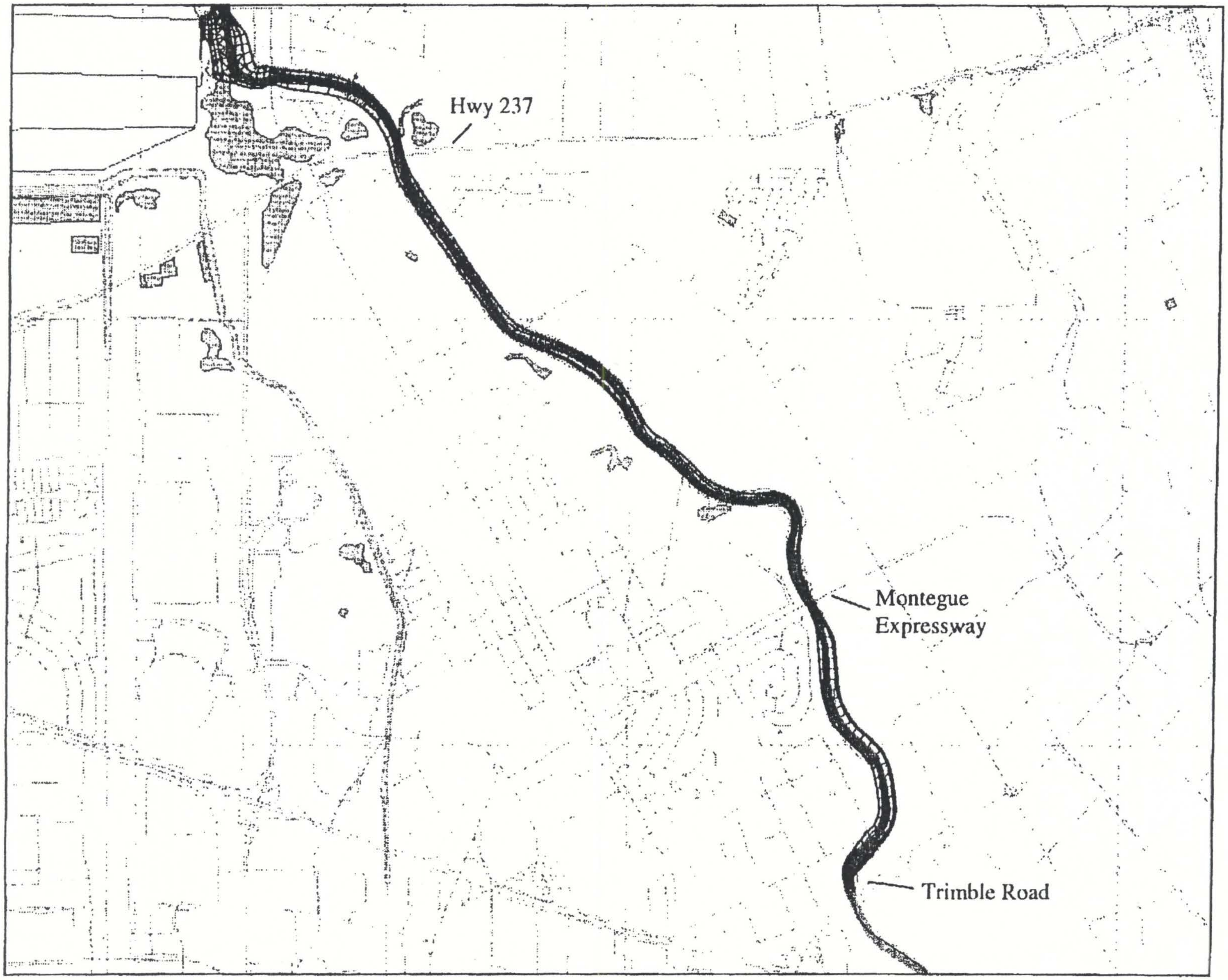


Figure 6.3. Detail of finite element network for Lower Guadalupe River, from Highway 237 to Trimble Road.

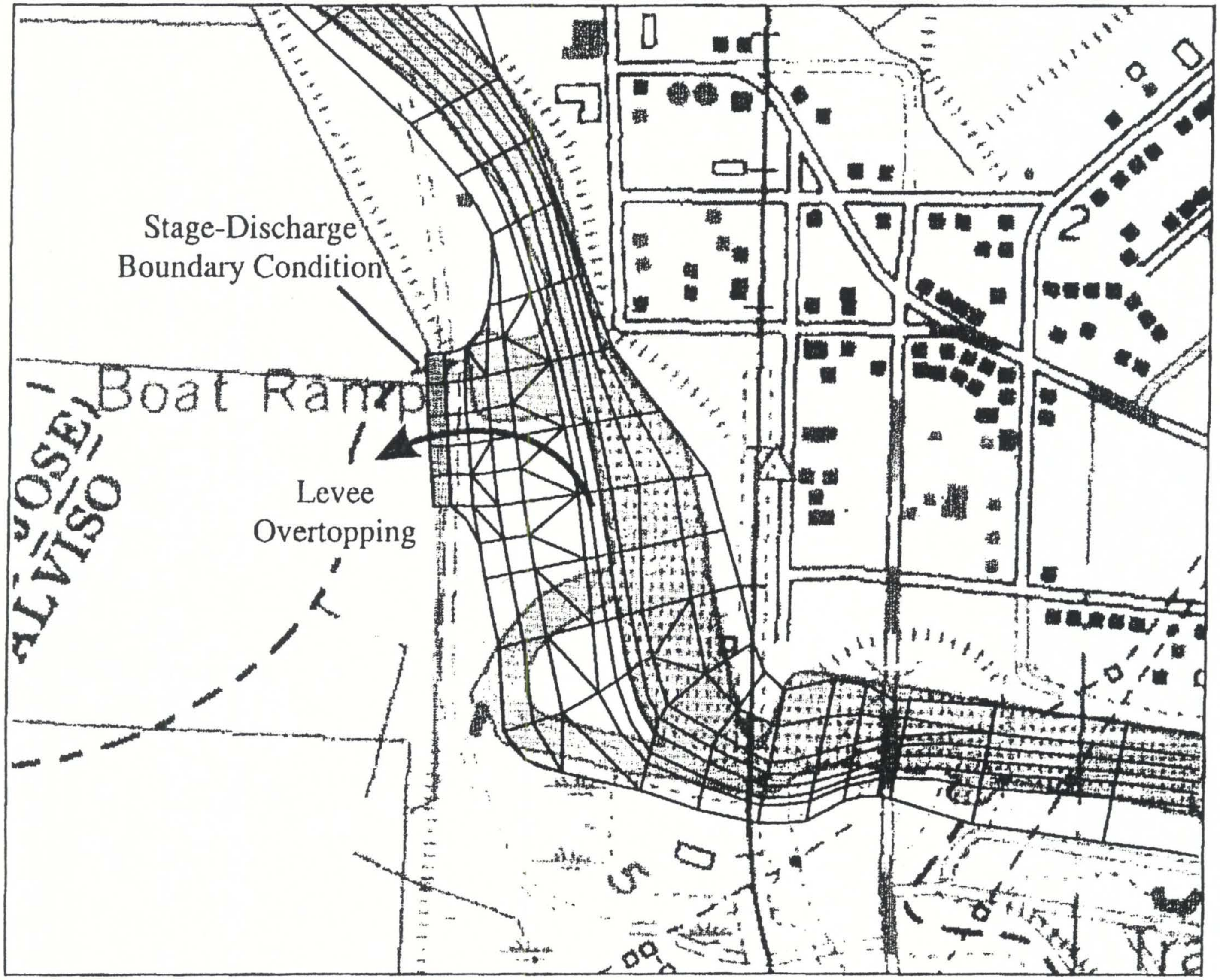


Figure 6.4. Location of Levee Overtopping, Stage-Discharge Boundary Condition at Cargill Salt Ponds.

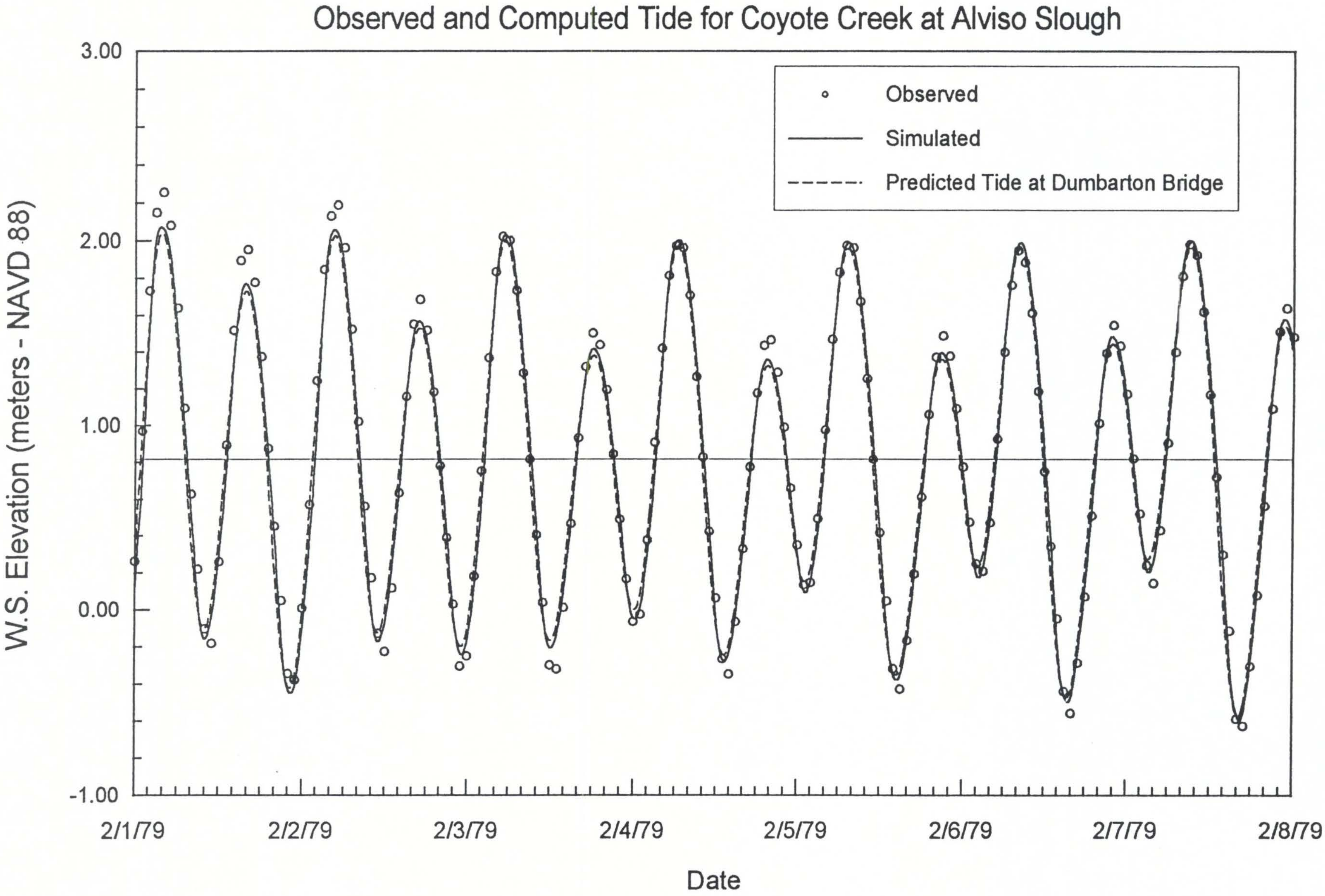


Figure 6.5 Simulated and observed water surface elevation for tide gauge 9414575 located in Coyote Creek at the confluence with Alviso Slough. The predicted tide at the Dumbarton Bridge is the simulation tidal boundary condition.

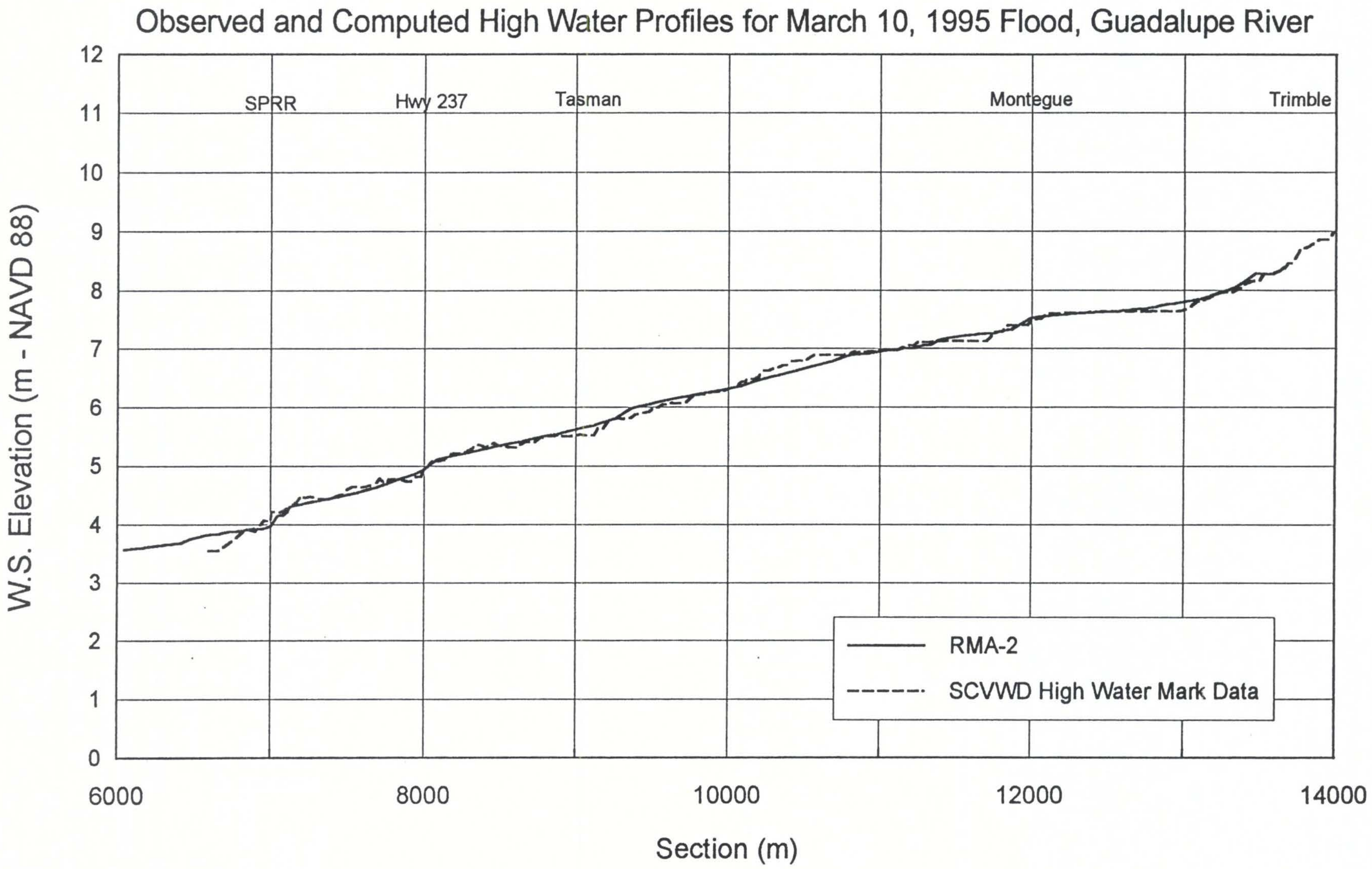


Figure 6.6 Observed and simulated maximum water surface profiles for March 10, 1995 flood event.

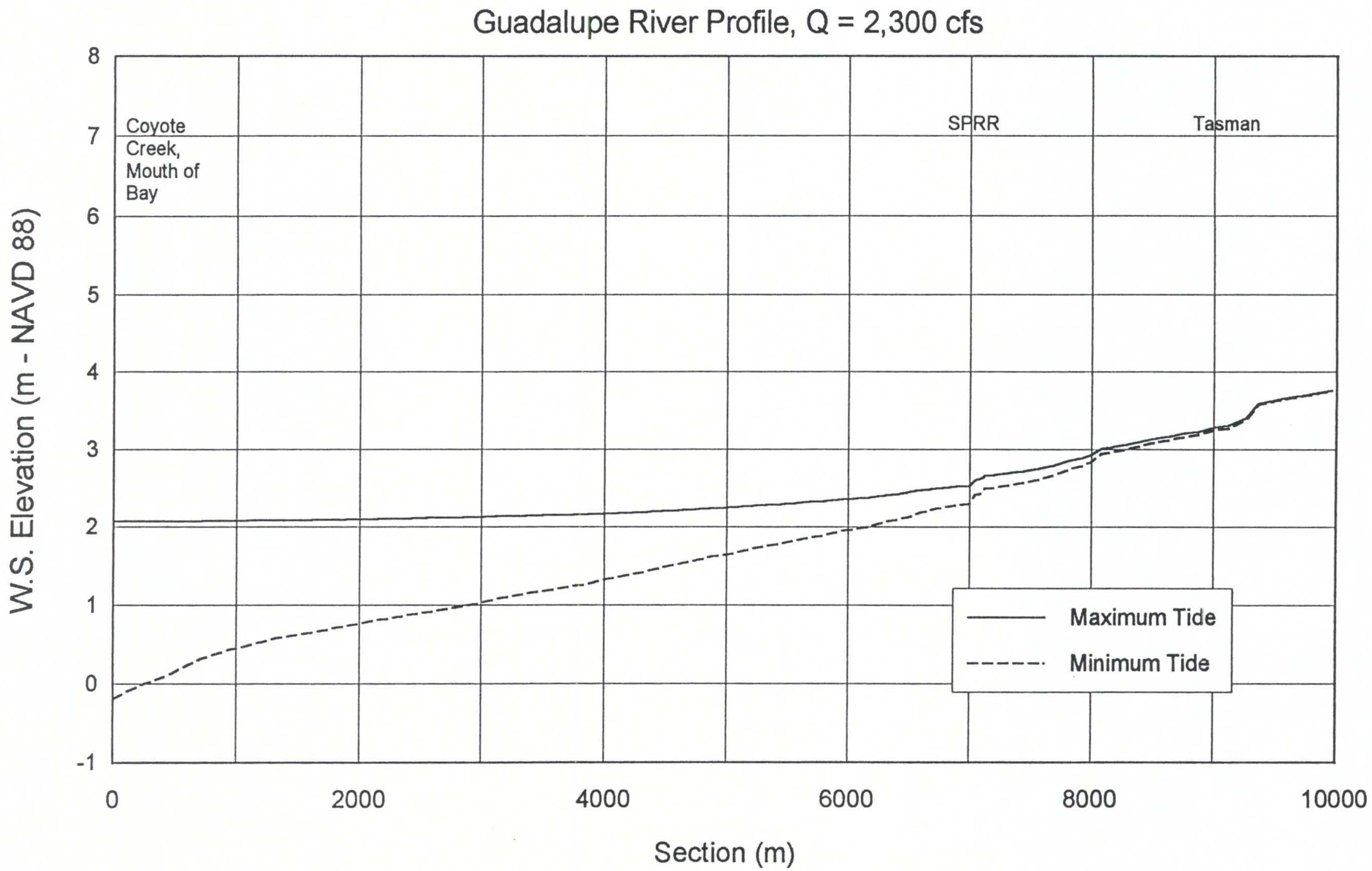


Figure 6.7 Guadalupe River water surface profiles at maximum tide for constant 2300 cfs discharge.

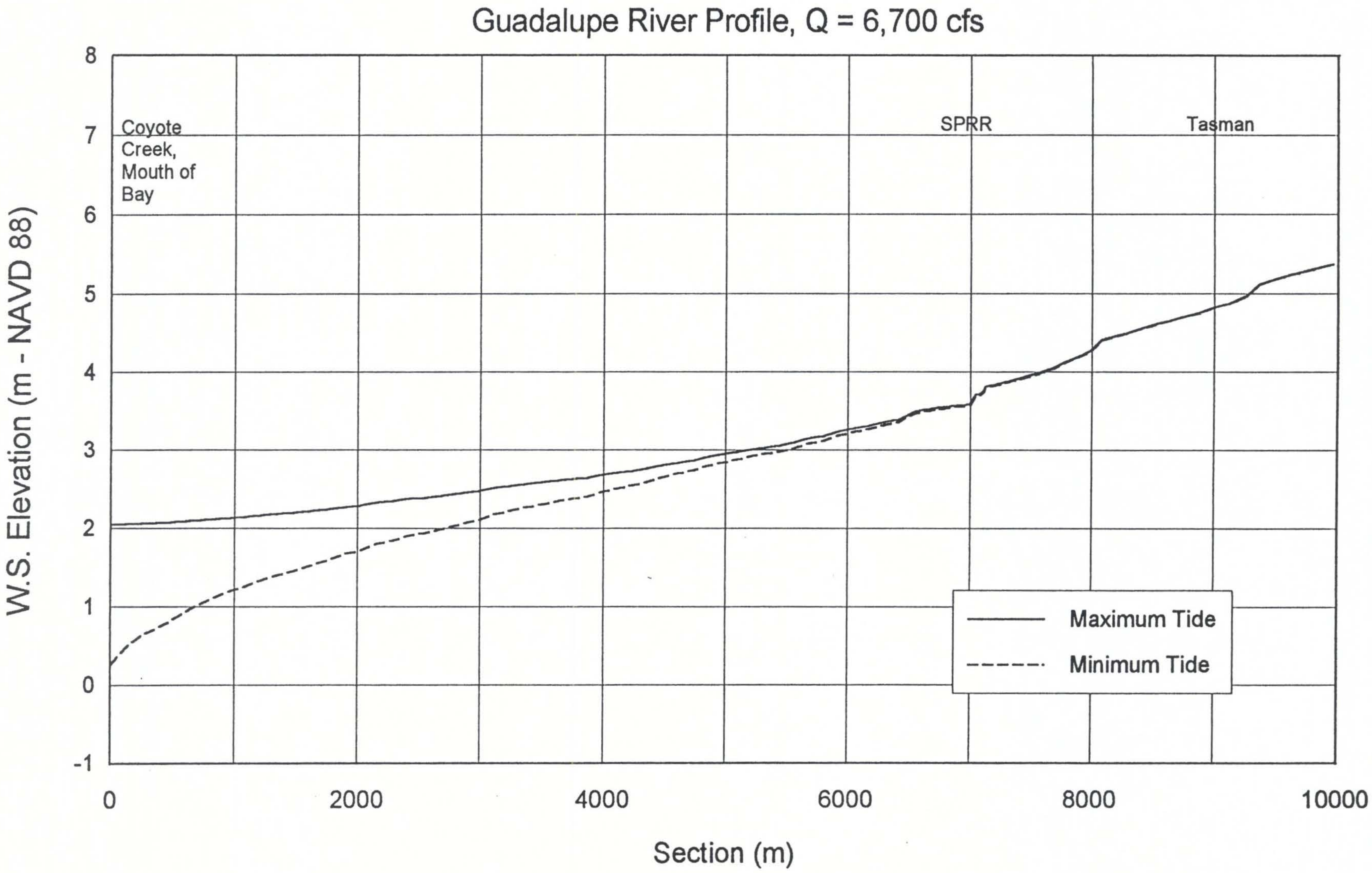


Figure 6.8 Guadalupe River water surface profiles at maximum and minimum tide for constant 6700 cfs discharge.

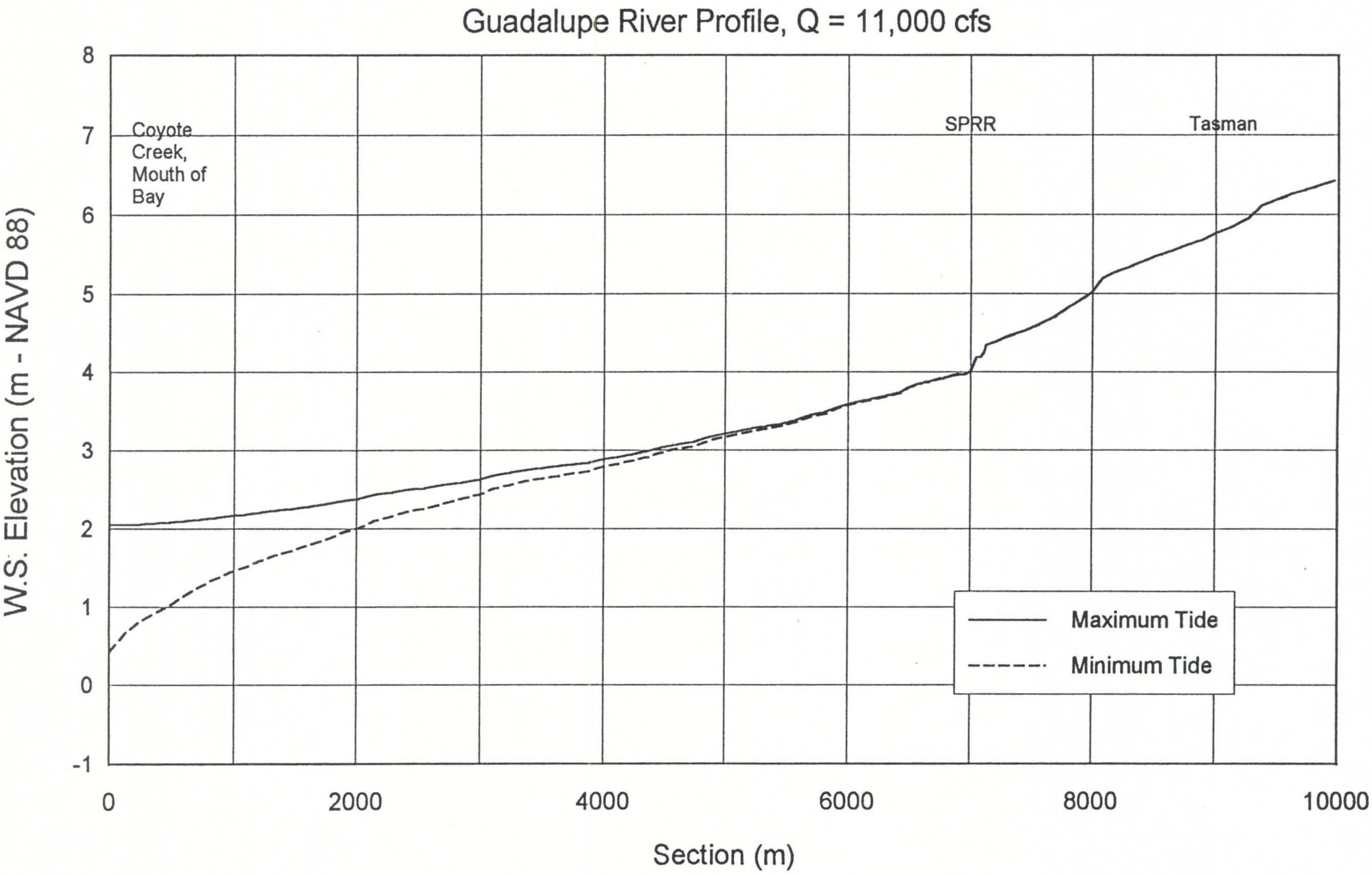


Figure 6.9 Guadalupe River water surface profiles at maximum and minimum tide for constant 11,000 cfs discharge.

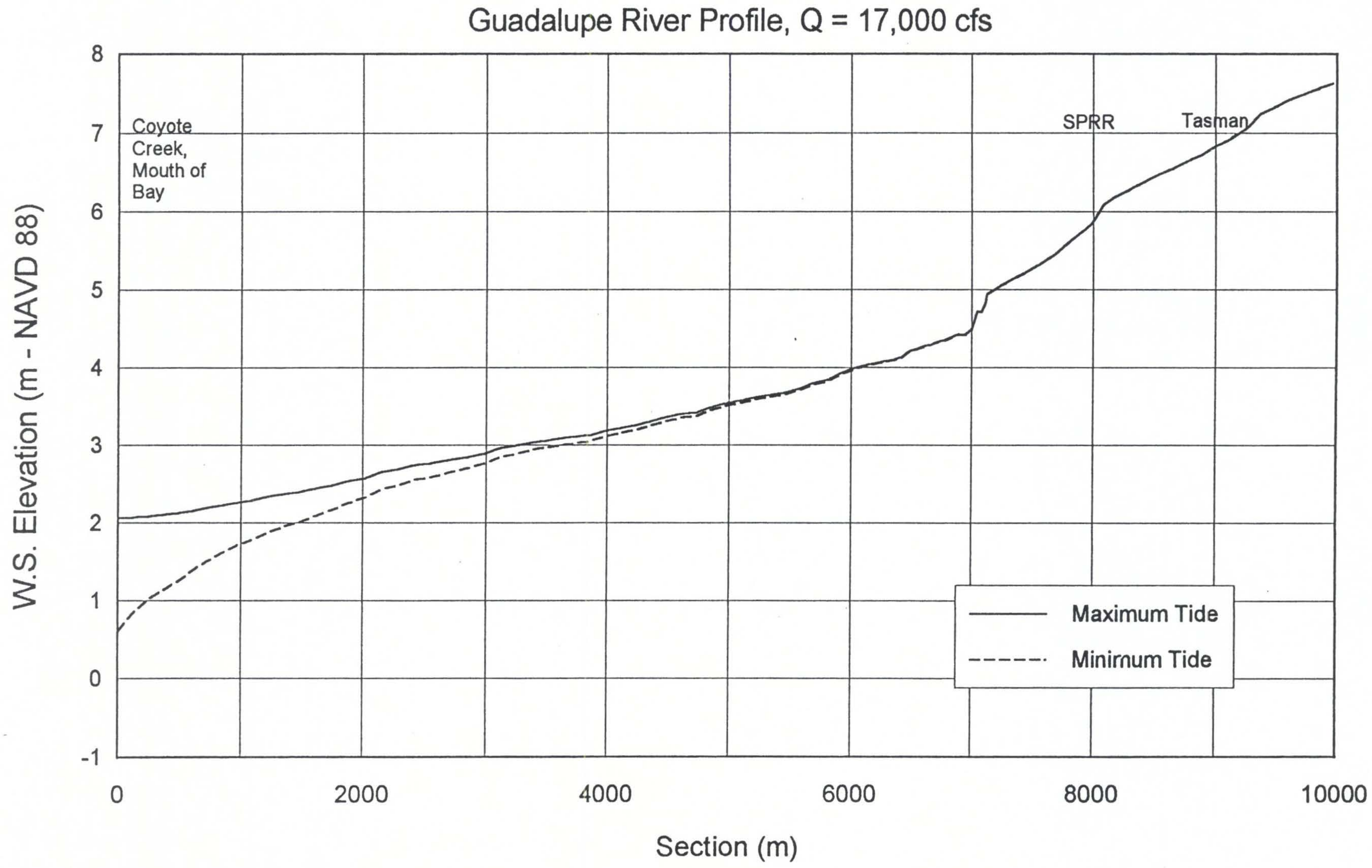


Figure 6.10 Guadalupe River water surface profiles at maximum and minimum tide for constant 17,000 cfs discharge.

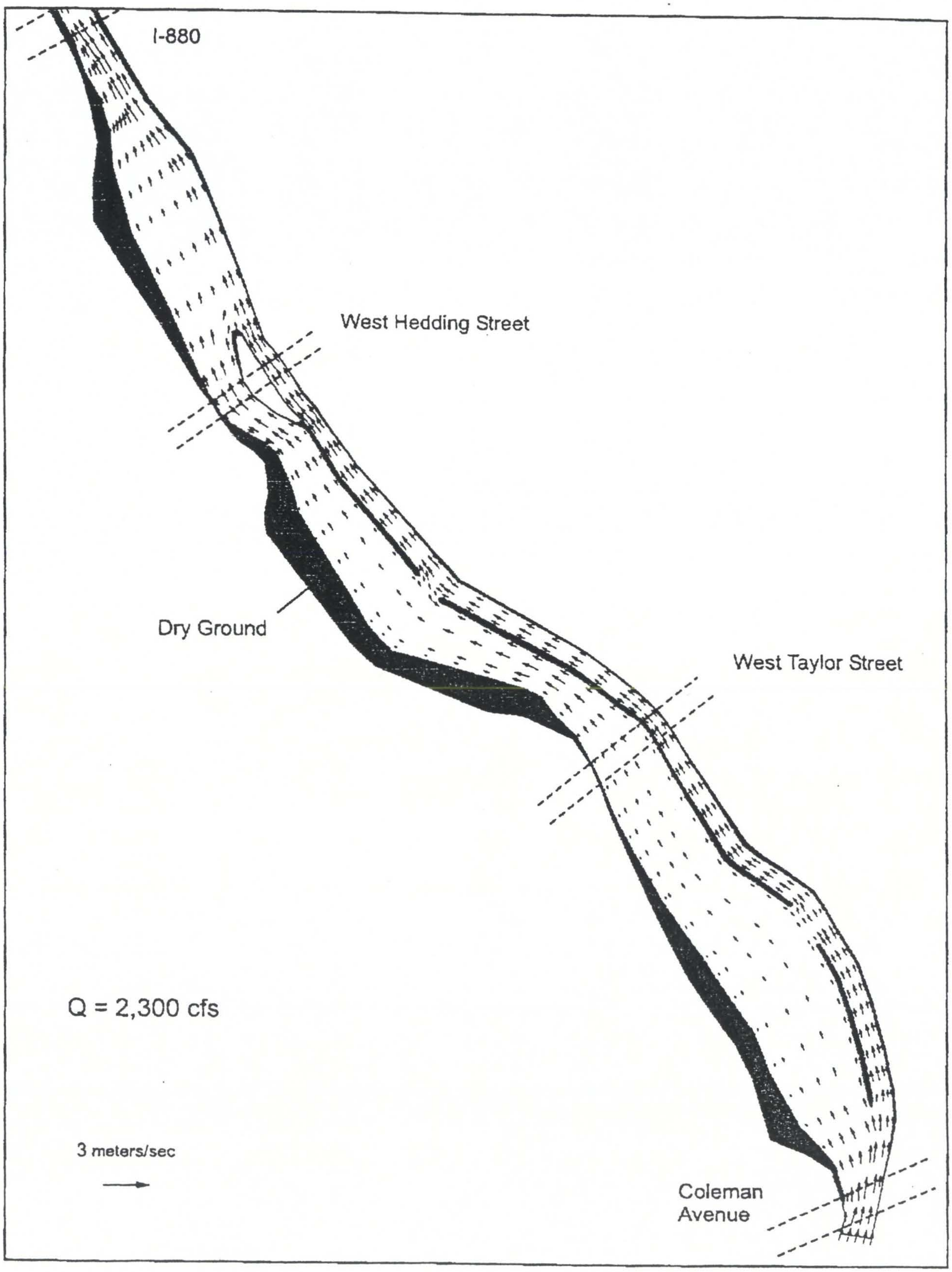


Figure 6.11. I-880 to Coleman Avenue, RMA-2 current velocity vectors for Q = 2,300 cfs.

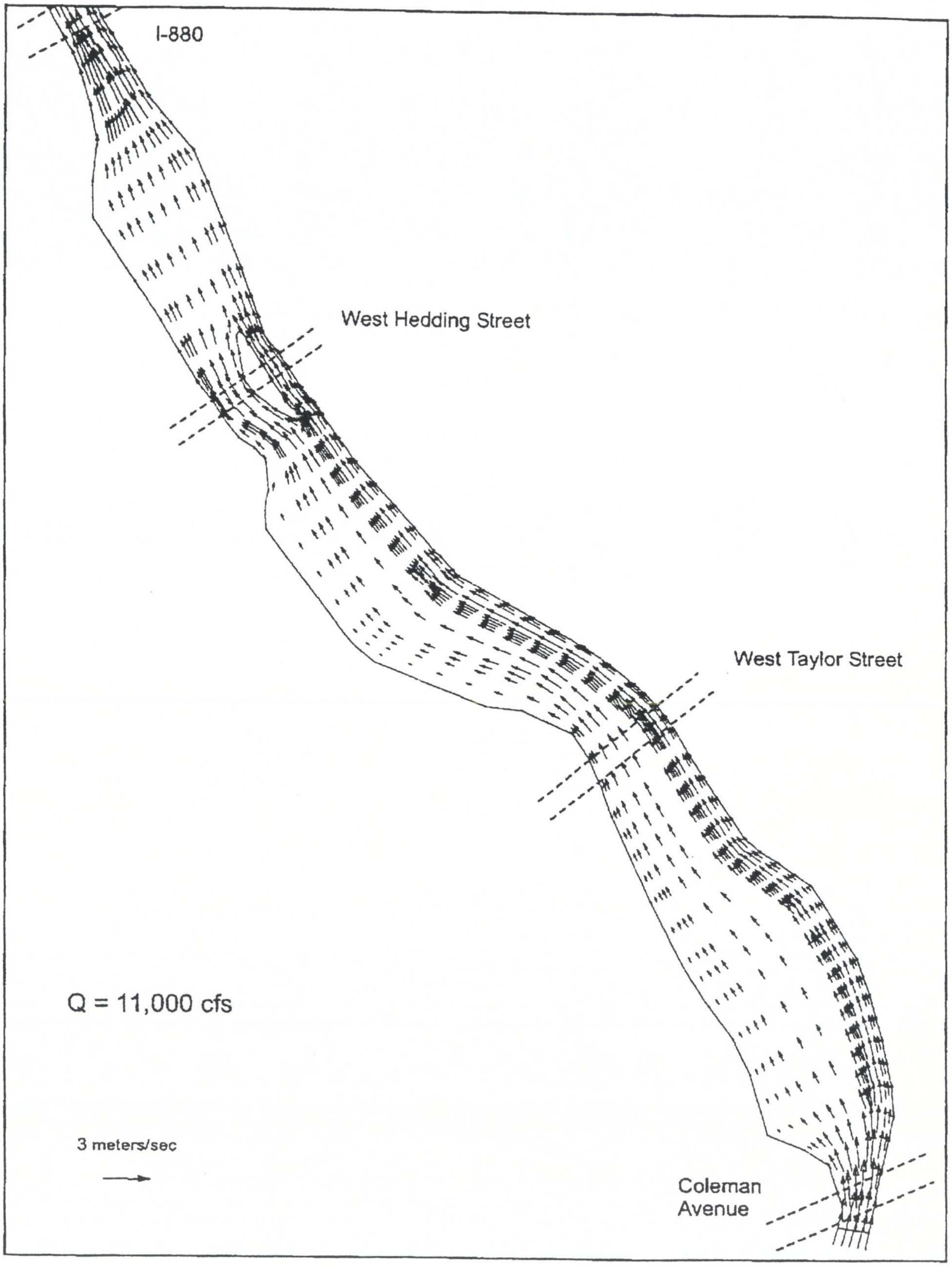


Figure 6.12. I-880 to Coleman Avenue, current velocity vectors for Q = 11,000 cfs.

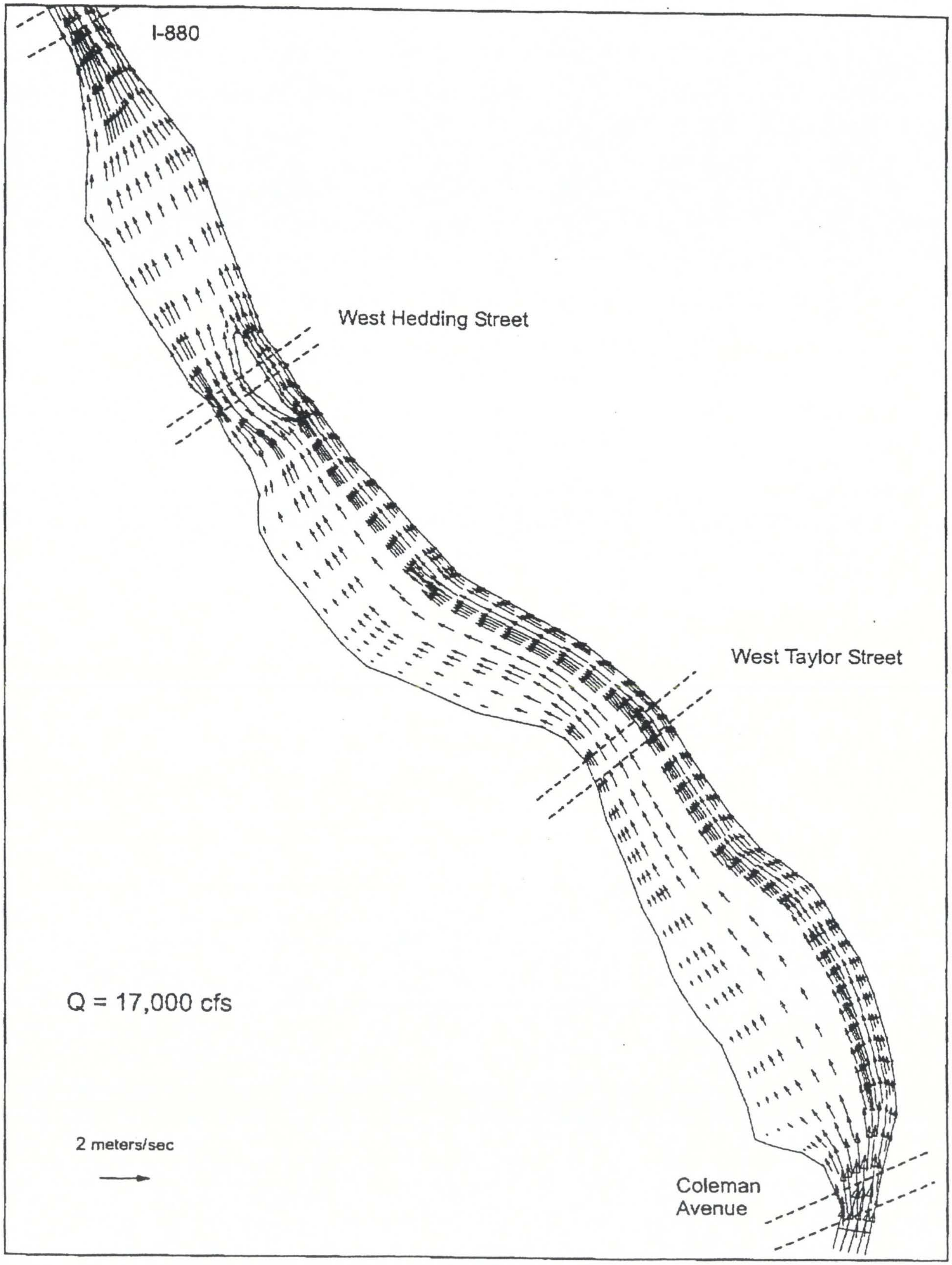
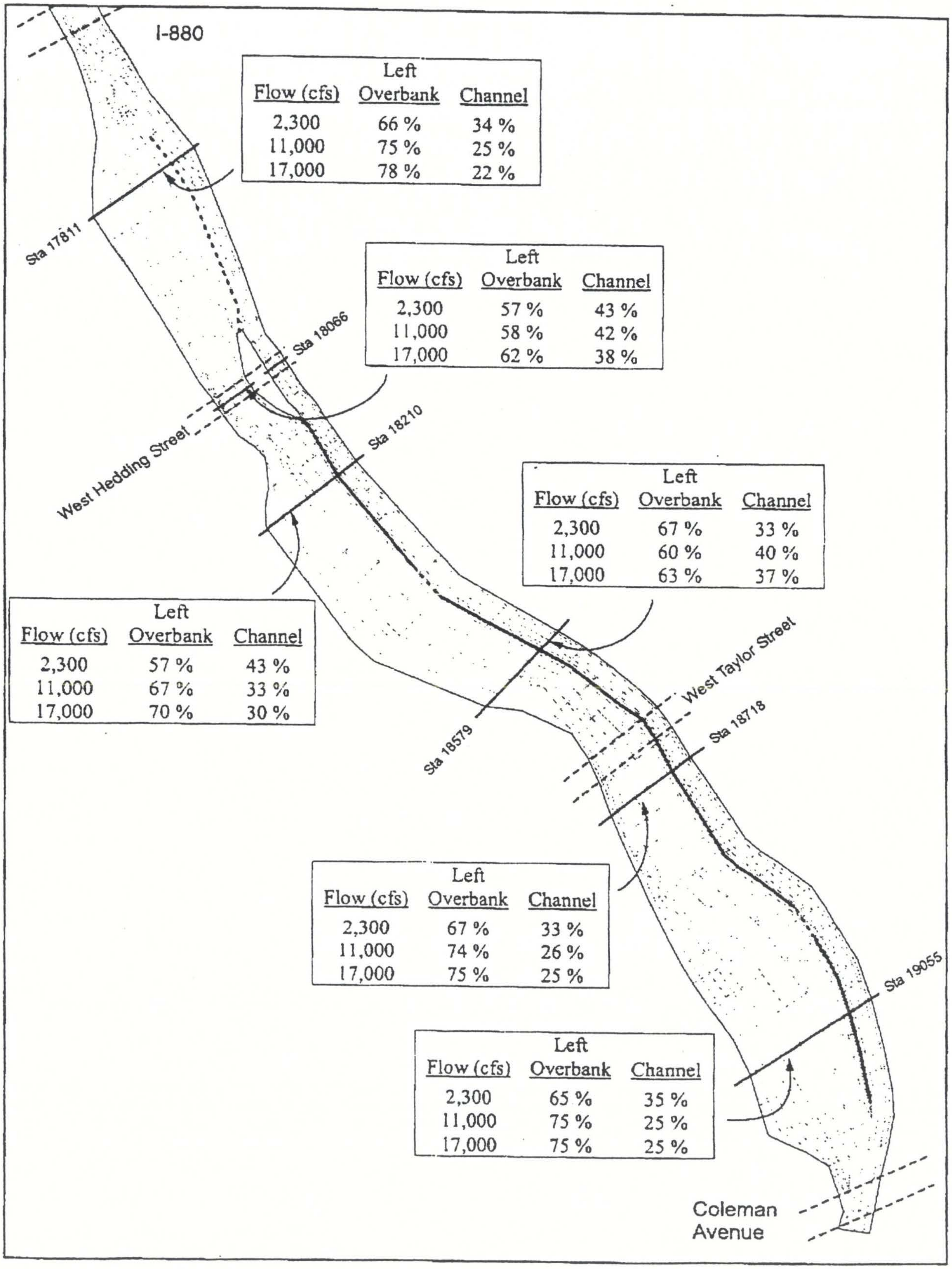


Figure 6.13. I-880 to Coleman Avenue, RMA-2 current velocity vectors for Q = 17,000 cfs.



	Left	
Flow (cfs)	Overbank	Channel
2,300	66 %	34 %
11,000	75 %	25 %
17,000	78 %	22 %

	Left	
Flow (cfs)	Overbank	Channel
2,300	57 %	43 %
11,000	58 %	42 %
17,000	62 %	38 %

	Left	
Flow (cfs)	Overbank	Channel
2,300	67 %	33 %
11,000	60 %	40 %
17,000	63 %	37 %

	Left	
Flow (cfs)	Overbank	Channel
2,300	57 %	43 %
11,000	67 %	33 %
17,000	70 %	30 %

	Left	
Flow (cfs)	Overbank	Channel
2,300	67 %	33 %
11,000	74 %	26 %
17,000	75 %	25 %

	Left	
Flow (cfs)	Overbank	Channel
2,300	65 %	35 %
11,000	75 %	25 %
17,000	75 %	25 %

Figure 6.14. I-880 to Coleman Avenue, RMA-2 derived flow splits.

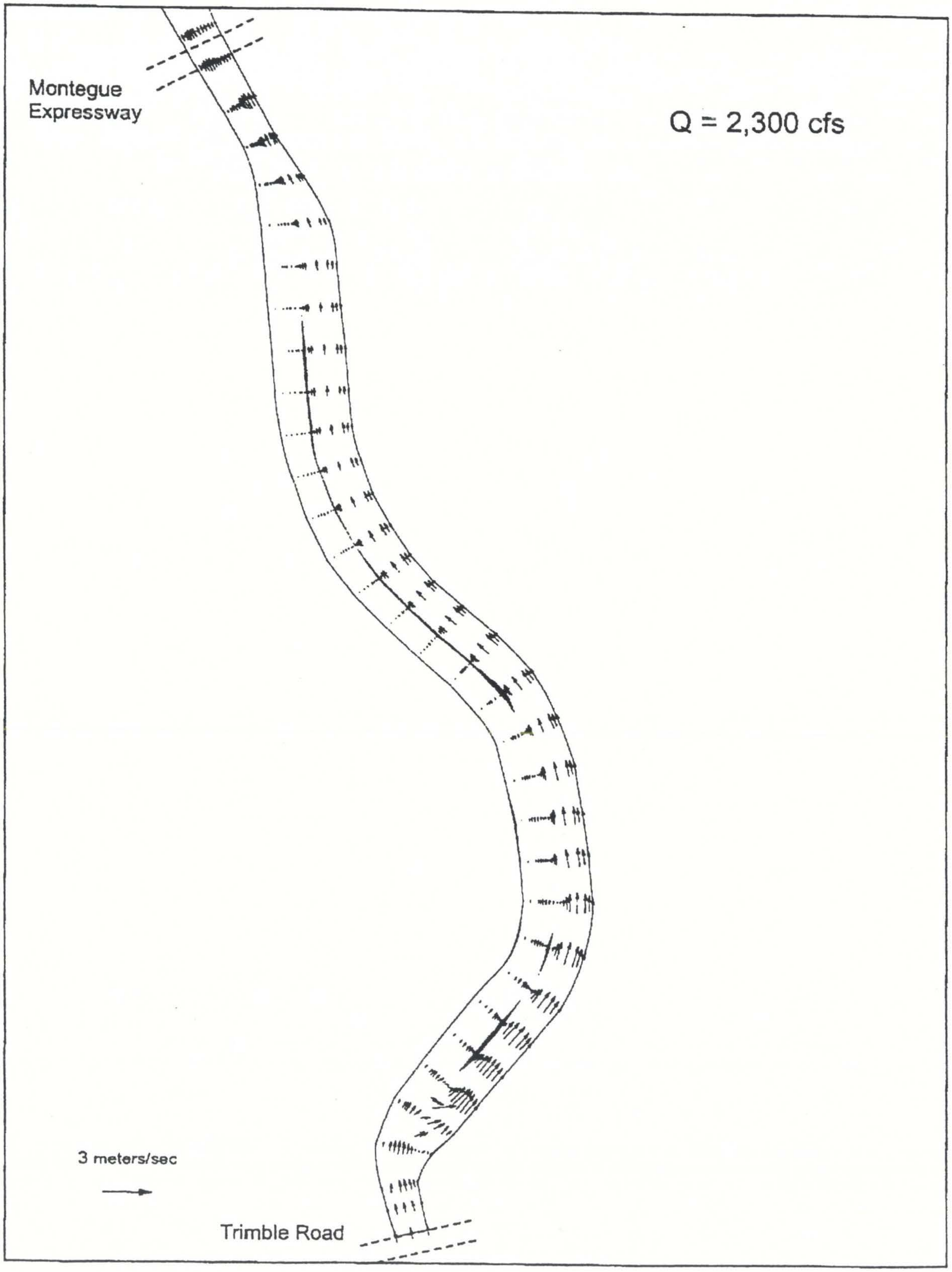


Figure 6.15. Montegue Expressway to Trimble Road, RMA-2 current velocity vectors for Q = 2,300 cfs.

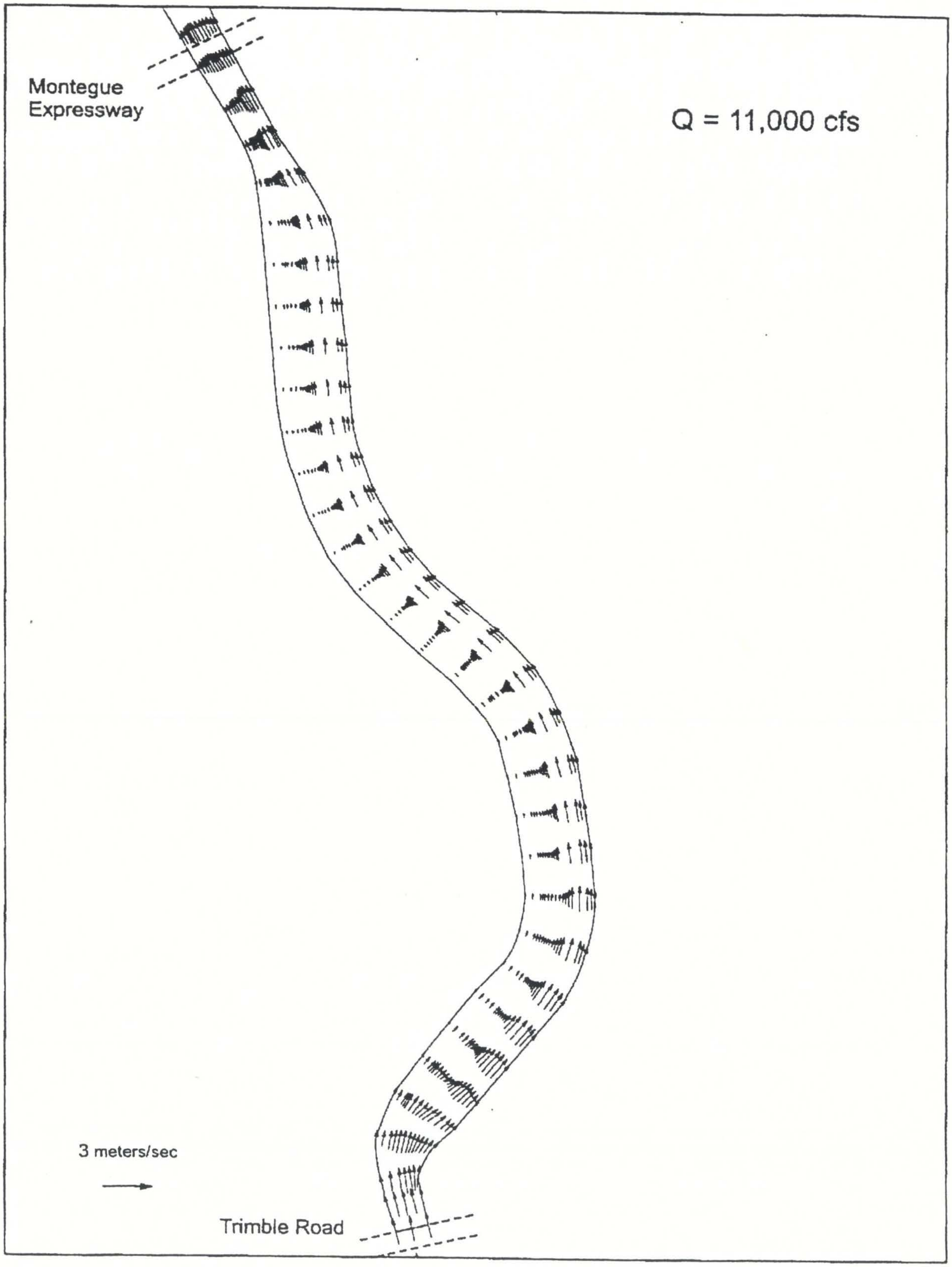
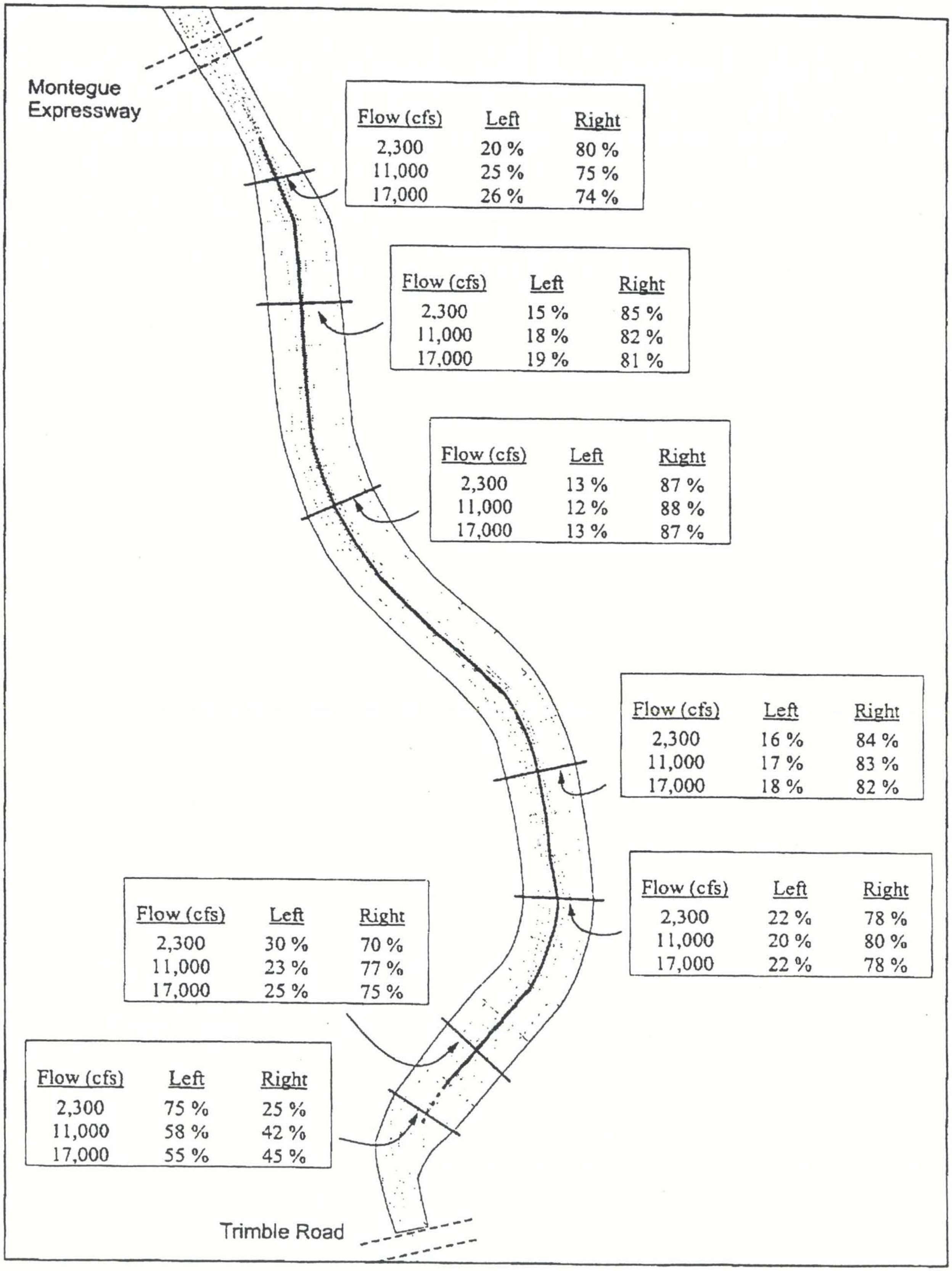


Figure 6.16. Montegue Expressway to Trimble Road, RMA-2 current velocity vectors for Q = 11,000 cfs.



Flow (cfs)	Left	Right
2,300	20 %	80 %
11,000	25 %	75 %
17,000	26 %	74 %

Flow (cfs)	Left	Right
2,300	15 %	85 %
11,000	18 %	82 %
17,000	19 %	81 %

Flow (cfs)	Left	Right
2,300	13 %	87 %
11,000	12 %	88 %
17,000	13 %	87 %

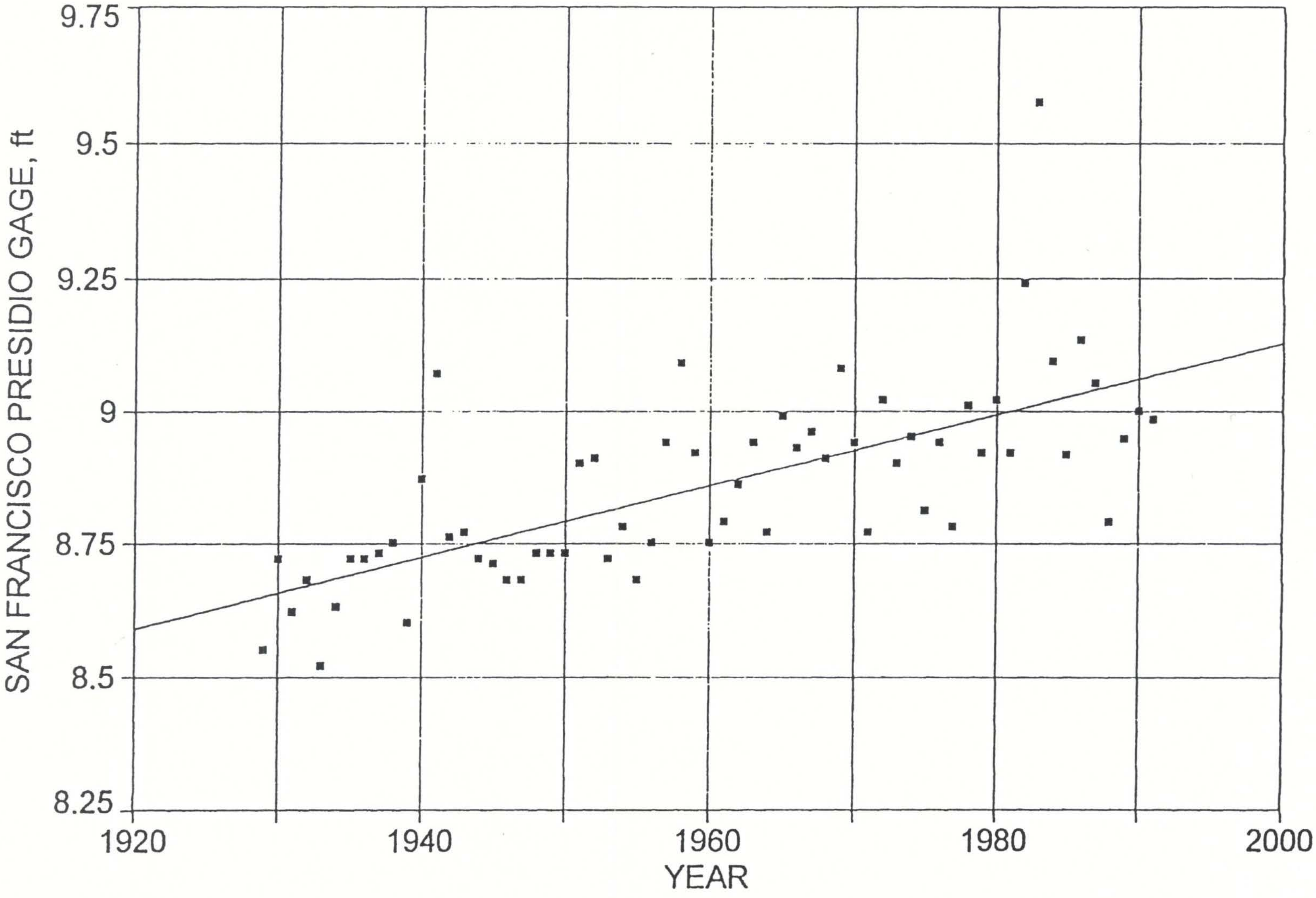
Flow (cfs)	Left	Right
2,300	16 %	84 %
11,000	17 %	83 %
17,000	18 %	82 %

Flow (cfs)	Left	Right
2,300	22 %	78 %
11,000	20 %	80 %
17,000	22 %	78 %

Flow (cfs)	Left	Right
2,300	30 %	70 %
11,000	23 %	77 %
17,000	25 %	75 %

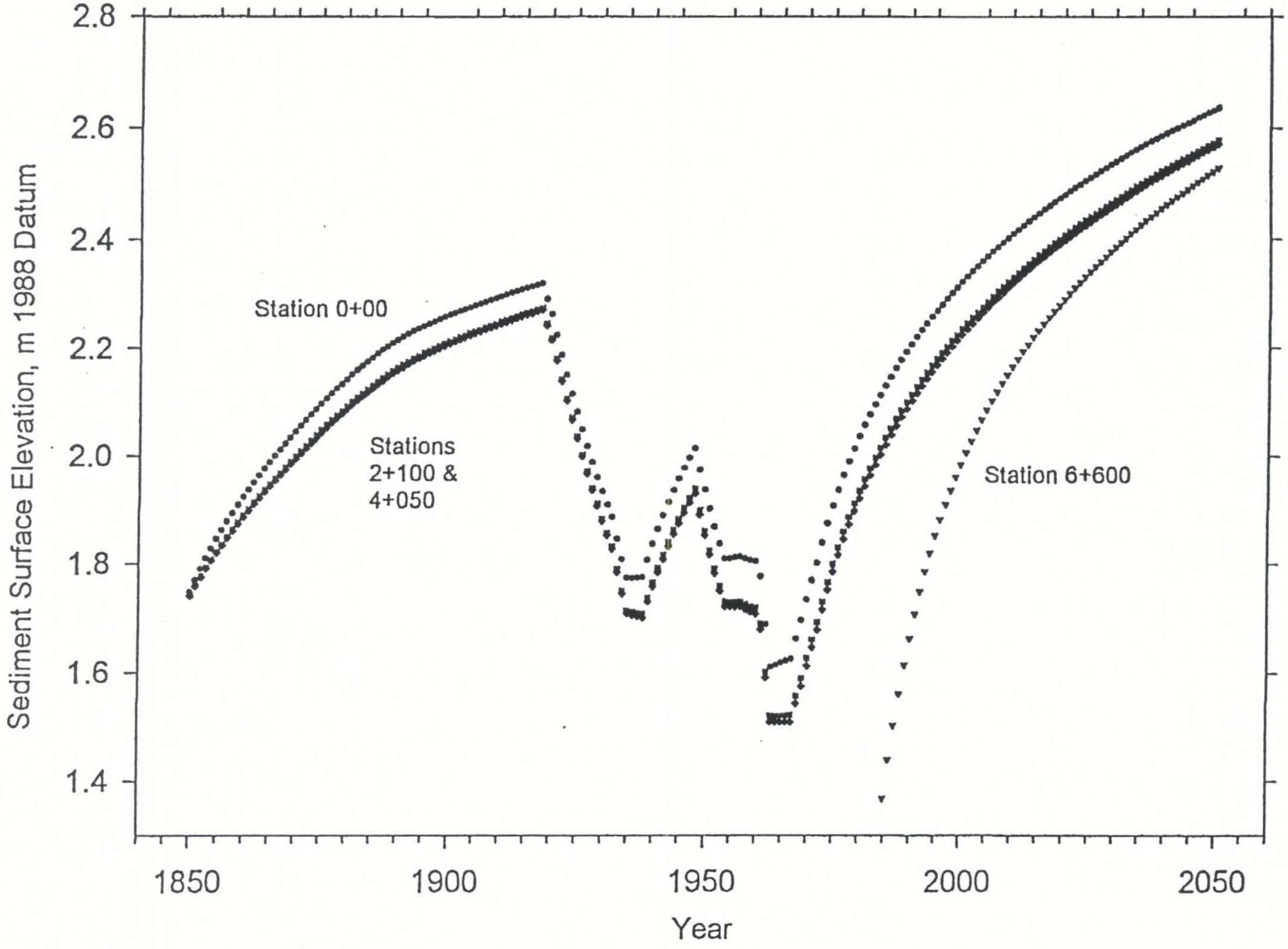
Flow (cfs)	Left	Right
2,300	75 %	25 %
11,000	58 %	42 %
17,000	55 %	45 %

Figure 6.17. Montegue Expressway to Trimble Road, RMA-2 derived flow splits.



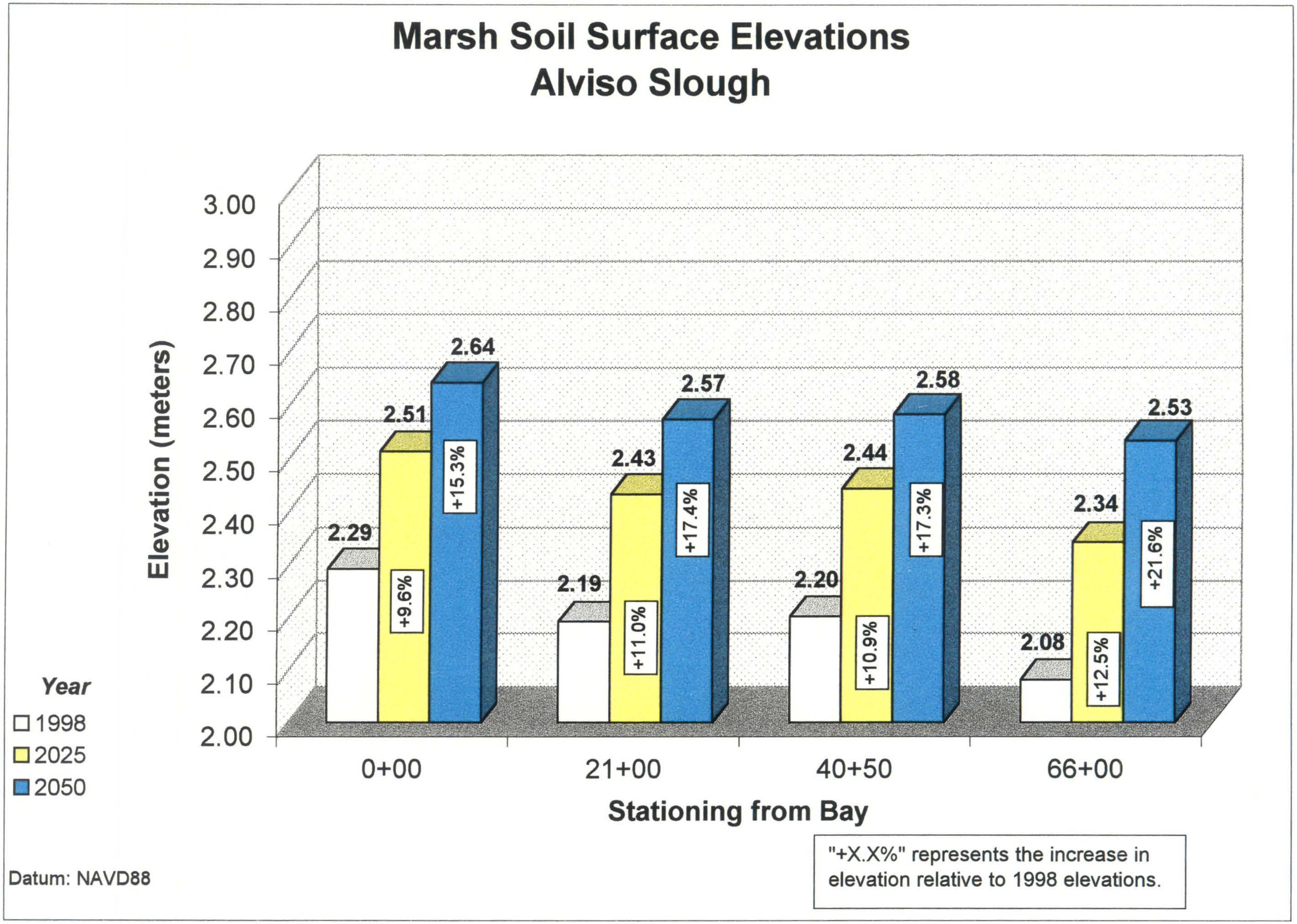
LINEAR FIT TO POST-1929 MEAN SEA LEVEL DATA

Figure 6.18



HISTORICAL AND PROJECTED ELEVATIONS AT MARSH STATIONS

Figure 6.19



Datum: NAVD88

"+X.X%" represents the increase in elevation relative to 1998 elevations.

accretion.xls

Figure 6.20

Chapter 7

River Hydraulics

7. River Hydraulics

The goal of the hydraulic analysis is to develop an understanding of the Lower Guadalupe River's flow characteristics and hydraulic sensitivity to changes in flow and other boundary conditions. The results of this analysis establish "baseline" hydraulic conditions necessary for the preliminary evaluation of project alternatives.

7.1. Approach

Existing hydrologic data prepared by the Corps of Engineers and the District along with existing HEC-2 models developed and utilized by the District were used in this analysis. The HEC-2 models were enhanced as necessary to evaluate existing hydraulic characteristics of the Lower Guadalupe River. Evaluations include a review of modeling boundary conditions, calibration to substantiate the model's suitability to represent baseline conditions, and the hydraulic analyses of the river for a variety of flow-frequency conditions. Results from these analyses are presented below.

7.2. Boundary Conditions

The following are significant boundary conditions which are necessary to perform the evaluation, and influence the results of the hydraulic modeling.

7.2.1. Hydrology and Flow Data

The following storm frequencies and associated peak discharge rates (Shown below in Table 7.1 in both System International units and English units) were used for the hydraulic analysis of the Lower Guadalupe River:

Table 7.1
Lower Guadalupe River Flow-Frequency Relationship[2]

Event Frequency	Peak Discharge (m^3/s)$^{(A)}$	 (cfs)$^{(B)}$	Comment(s)
2-Year	65.14	2,300	
5-Year	127.44	4,500	
10-Year	189.74	6,700	
25-Year	283.20	10,000	
—	311.52	11,000	10 March 1995 Storm Event
50-Year	382.32	13,500	
100-Year	481.39	17,000	

(A) – units are cubic meters-per-second
(B) – units are cubic feet-per-second

These values are assumed to include local drainage inflows from city/county pump stations discharging into the Guadalupe River at various points within the Project. Figure 7.1 depicts the flow-frequency relationship, and includes both the March 1995 event plus data collected at the San Jose gaging station (USGS gage 11169000) during the period 1930 to 1995. Figure 7.2 shows Guadalupe River annual peak discharges for 1930 through 1997. The mean annual peak discharge for this period is approximately 98.53 m^3/s (3,479 cfs).

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[2] U.S. Army Corps of Engineers, *"Santa Clara County, California, Guadalupe River and Coyote Creek Hydrologic Engineering Office Report,"* July 1977.

7.2.2. Channel Geometry

The District provided NHC with several HEC-2 models (tabulated below) for use in this analysis. The models cover the Guadalupe River from I-880 downstream to the Bay. COE HEC-2 models for the reach from I-880 upstream were also acquired to ensure that: 1) the geometry for the channel was contiguous between the District's and COE's models, and 2) upstream boundary conditions (design stage, cross-section, velocities, etc.) matched COE designs implemented in the Contracts 1 & 2 reaches.

The HEC-2 models used as the basis for starting this analysis were:

Model:	**Source:**	**Model Coverage:**
Exist-R0.dat	District	Bay upstream to SPRR
Exist-R1.dat	District	SPRR to Tasman Dr.
Exist-R2.dat	District	Tasman Dr. to Montague Expressway
Exist-R3.dat	District	Montague to HWY 101
Exist-R4.dat	District	HWY 101 upstream to I-880
Ctrns.dat	District	CalTrans Mitigation Reach (15+570 → 17+520)
C2-CHGRD.dat	COE	I-880 and upstream

In summary, preparation for hydraulic analyses included the following exercises:

- Using existing HEC-2 cross-sections provided by the District,the C2-CHGRD.DAT Corps' HEC-2 model was used to represent the upstream changes as a result of the COE Contract No. 2.
- Preliminary CalTrans mitigation plans, provided by the District, were represented in the model between stations 14+501 and 17+580.
- Bank stations in the District's model from levee tops to top of low flow channel were redefined
- The HEC-2 deck was converted from Metric to English units for use in HEC-6
- The new Skyport bridge crossing was included in the HEC-2 model.

7.2.3. Bridges

There are twelve (12) bridges crossing over Guadalupe River throughout the project study area. Those bridge crossings are:

- UPRR Bridge near Alviso
- Gold Street
- Highway 237 (East-bound)
- Highway 237 (West-bound)
- Tasman Drive
- Montague Expressway
- Trimble Road
- Highway 101
- Airport Parking Lot Bridge
- Airport Parkway
- Skyport Road (under constr.)
- I-880

All of these bridges except I-880 were included in the HEC-2 modeling to more accurately depict their hydraulic influences to the overall water surface profile, as well as to gain an understanding of their impacts with respect to velocities, energy slopes, depths, etc.

Additional HEC-2 model preparation consisted of:

- Changing non-pressure flow bridges from Special Bridge to Normal Bridge.
- Modifications of the contraction and expansion coefficients from those in the District models. The contraction coefficients were changed to 0.1 for uniform channel reaches and 0.3 for abrupt transitions at bridges. The expansion coefficients were changed to 0.3 for smooth channel reaches and 0.5 for abrupt transitions at bridges.

7.2.4. Roughness Coefficients

The District provided NHC with channel roughness values for the models. The District utilized a single composite "n" value for the entire cross section. Table 7.2 below shows the composite "n" values by cover type provided to NHC by the District.

Table 7.2
Single, Composite Mannings "n" Values[A]

Description	N Value		Note
	Case #1	Case #2	
Maintenance Road	0.028		
Concrete Sack	0.02		
Concrete	0.015		
Tall Trees	0.07		Not used in the model
Trees	0.06		
Willows	0.06		
Brush Tree	0.045		Not used in the model
Dense Heavy Tree/Dense Willows	0.125		
Mixture of Willows and Trees	0.125		
Brushes/Heavy Brushes	0.045		
Pond	0.03		
Channel Under Water	0.025		
Dry woody Type	0.035		
Grass on Slope	0.03		
Long Tules	0.05		
Short Tules	0.04		
Heavy Mixtures of Tules/Vegetation	0.08	0.2	case 2 - value is forced to match HWM
Mixtures of Tules	0.055		
Mixtures of Vegetation	0.045	0.1	case 2 - value is forced to match HWM
Light Mixtures Vegetation	0.04		
Light Grass	0.02		Values from this line and below
Fresh Water Vegetation/Grass	0.02		seem lower than they normally would
Channel with Gravel Bed	0.015		be. But those values are calibrated
Gravel Bed	0.018		to match the high water marks
Lighter Mixture of Vegetation	0.02		surveyed in Reach A.
Short Grass	0.02		
Earth Cut	0.025		
Very Light Mixture of Vegetation	0.015		

(A) – District "n" values based on observed vegetation and high water marks for March 1995 event.

For the purpose of gaining more control over cross-section specific refinement, NHC took the composite "n" HEC-2 models, combined them into a single HEC-2 model covering the entire project reach (from SPRR upstream to I-880) and converted the roughness coefficients over to separate values for overbanks and for the main channel. This was done in order to match computational procedures used in the computer model HEC-6T, and to gain more flexibility in simulating dramatic changes in ground cover type across individual cross sections, and allow for more accurate cross-sectional representation when calibrating the model.

In summary, the channel roughness baseline conditions for analysis were as follows:

- Roughness was changed from a single composite roughness for the channel to a main channel and two overbank areas.
- Roughness along the left and right overbanks ranged from 0.03 in lightly vegetated reaches to 0.8 in densely vegetated reaches. Channel roughness ranged from 0.03 in the lightly vegetated reaches to 0.2 in densely vegetated reaches.
- Future "without-project" channel roughnesses were assumed to be the same as existing roughnesses.

7.3. Hydraulic Analyses

The hydraulic analysis consisted of first calibrating the HEC-2 model for existing conditions. Then the model was run under various hydrologic event scenarios to develop baseline hydraulic characteristics of the Project reach for a range of hydrologic events including the 2-, 5-, 10-, 25-, 50-, and 100-year events.

7.3.1. Calibration

The HEC-2 model was calibrated to the 10 March 1995 storm event which was the recent flood of record. Available information and measured data for the 1995 flood event from aerial photos, high water marks, and estimated flood depths at structures (buildings, bridges, etc.) were used during model calibration.

For the purposes of this study, high water marks and aerial photograph-derived water surface profile elevations for the March 1995 event were provided to NHC by the District. High water data were provided from approximately river station 6+600 (near Alviso) upstream throughout the entire project study area to I-880 at river station 17+612. A plot of the high water profile is provided as Figure 7.3. The estimated peak discharge rate of 311.52 m^3/s (11,000 cfs) for the March 1995 event was used in the calibration exercise.

The HEC-2 model was adjusted by modifying Manning's "n" roughness coefficients until water surface elevations at the modeled cross-sections closely matched observed high water elevations at the same cross-sections. The revised "n" values were compared against the cover type representation (see Table 7.2, Section 7.2.4) to verify that the adjusted values were still reasonable representations of physical conditions at that site. Figure 7.4 compares the resulting calibrated HEC-2 model water surface profile against the observed March 1995 flood profile.

7.3.2. Hydraulic Modeling

Hydraulic characteristics in the project study area for 2-, 5-, 10-, 25-, 50- and 100-year events were evaluated as well as those of the March 1995 event.

Figure 7.5 shows the water surface profile relationships for the 10 March 1995 flood and the Corps of Engineers 100-year flood event surfaces relative to both left and right channel banks as well as the twelve bridge crossings. (The blue rectangles depict bridge crossings including deck and low chord elevations as well as bridge deck width.) It is important to note that both the bridge soffit and the deck elevations are shown relative to the water surface profiles (the top and bottom sides of the blue rectangles). Of significant importance is the fact that the 100-year storm water surface exceeds the bankline profiles approximately 400 meters upstream of Montague Expressway. It is also important to note that constrictions at several of the bridges (SPRR, HWY 237, Tasman Drive, Montague Expressway, HWY101, and Airport Parkway) noticeably influence the river hydraulics. In addition, it is likely that during a flood event close to (and/or exceeding) the 100-year event, four of the bridge crossings will undergo pressure flow (HWY 237 East, HWY 237 West, Tasman, Montague). Montague Expressway and HWY 237 East may actually experience overtopping. Figure 7.6 depicts the 50-year event water surface profile and the channel invert in addition to the information presented in Figure 7.5.

A review of Figure 5.9 in Chapter 5 reveals a fairly distinct change in the stream's invert slope over the entire length of the project study area. Notice the steeper overall bed slope from about Montague Expressway upstream past I-880, which results in a steeper energy gradient. This stretch has higher velocities during high flows and a greater propensity to transport materials through the stream system relative to the portion of the study area from Montague Expressway downstream to, and past the SPRR crossing near Alviso. This lower reach with its milder slope is more likely to experience sediment accumulation or aggradation. (A more thorough analysis of the sediment transport potential is presented later in Chapters 8 through 11.)

Figure 7.7 shows the relationship between the HEC-2 March 1995 simulation, the observed March 1995 high water profile, and the main channel's average flow velocity. Notice the distinctive velocity increases occurring at bridge locations as the flow is constricted and rapidly progresses through the bridge sections. This localized high velocity condition can increase during larger events or when debris is available to collect on bridge piers and/or associated utilities attached to bridges. Accumulating debris further decreases the cross-sectional area necessary to convey the flood flows past the bridge, resulting in a greater increase in water surface elevation upstream of the bridge.

Hydraulic characteristics computed for the project study area are shown in Figures 7.8 through 7.11 for the range of flows from a 2-year to 100-year event. For these diagrams, the abcissa is defined by reach number rather than channel stationing as shown on previous figures. The reaches are defined as follows:

Reach	Boundaries	Reach	Boundaries
1	Co. Marina – SPRR	9	HWY 101 – Airport Pkwy
2	SPRR – HWY 237	10	Airport Pkwy – 16+800
3	HWY 237 – Tasman Dr.	11	16+800 – I-880
4	Tasman Dr. – H.Hetchy	12	I-880 – Hedding
5	H.Hetchy – Montague	13	Hedding – Taylor
6	Montague – 12+900	14	Taylor – D/S Coleman
7	12+900 – Trimble Rd.	15	Upstream of Coleman
8	Trimble Rd. – HWY 101	16	USGS Gage

- Figure 7.8 shows the relationship between "Energy Slope" versus reach (channel distance) for the suite of flow-frequency events (2- through 100-yr storms). Notice the similarity in changes to energy slope for the regime upstream of Montague Expressway relative to the regime downstream of Montague. This is consistent with the observations depicted in Figure 5.9 (see Chapter 5).

- Figure 7.9 shows the relationship of flow depth versus reach location (distance) for the suite of flow-frequency events. As expected, the overall depth increases for the region from about Trimble road downstream, which is consistent with the portion of the channel where the overall bed slope is more mild.

- Similarly, Figure 7.10 shows the relationship between flow topwidth and reach location for the suite of flow-frequency events. The fact that the lines are all quite close together indicates that most of the flow regimes are probably inundating the channel cross-sections from right bank levee to left bank levee.
- Lastly, Figure 7.11 presents average flow velocity versus reach location for the suite of flow-frequency events. Highest velocities typically occur near existing bridges.

7.4. Conclusions of Hydraulic Modeling

The objective of the hydraulic analysis was to summarize hydraulic characteristics of the Lower Guadalupe River for present conditions. This has been accomplished by calibrating the HEC-2 model to a recent, well documented flood event (the 10 March 1995 storm), and then applying the model to simulate the river's response to a full range of flows.

The following conclusions were made based on results of the HEC-2 analysis:

- Portions of the Lower Guadalupe River cannot convey the 100-year flood event in channel. As Figures 7.5 and 7.6 demonstrated, the river loses containment of the 100-year event approximately 400 meters upstream of Montague Expressway. From this point and downstream, the 100-year event water surface profile appears to exceed the elevations of much of the left and right levees.
- Certain bridges impose significant, localized hydraulic constraints during (at or near) the 100-year event. Four of the bridges, HWY-237 West, HWY 237-East, Tasman Dr., and Montague Expressway all appear to be approaching pressure flow. In addition, HWY 237-East and Montague Express Way, may be overtopped during the 100-year (or greater) event. Debris loading can substantially worsen hydraulic conditions at many of the bridges including those previously cited.
- Regular channel maintenance is important in order to maintain (at least) the present channel capacity in the Lower Guadalupe River. An increase in sediment deposition or increase in vegetative growth within the levees will worsen the hydraulic conditions and further limit hydraulic capacity.
- Annual monitoring during and after flood events will provide critically needed data for further refinement of the HEC-2 models and of our understanding of the hydraulic characteristics of the Lower Guadalupe River.

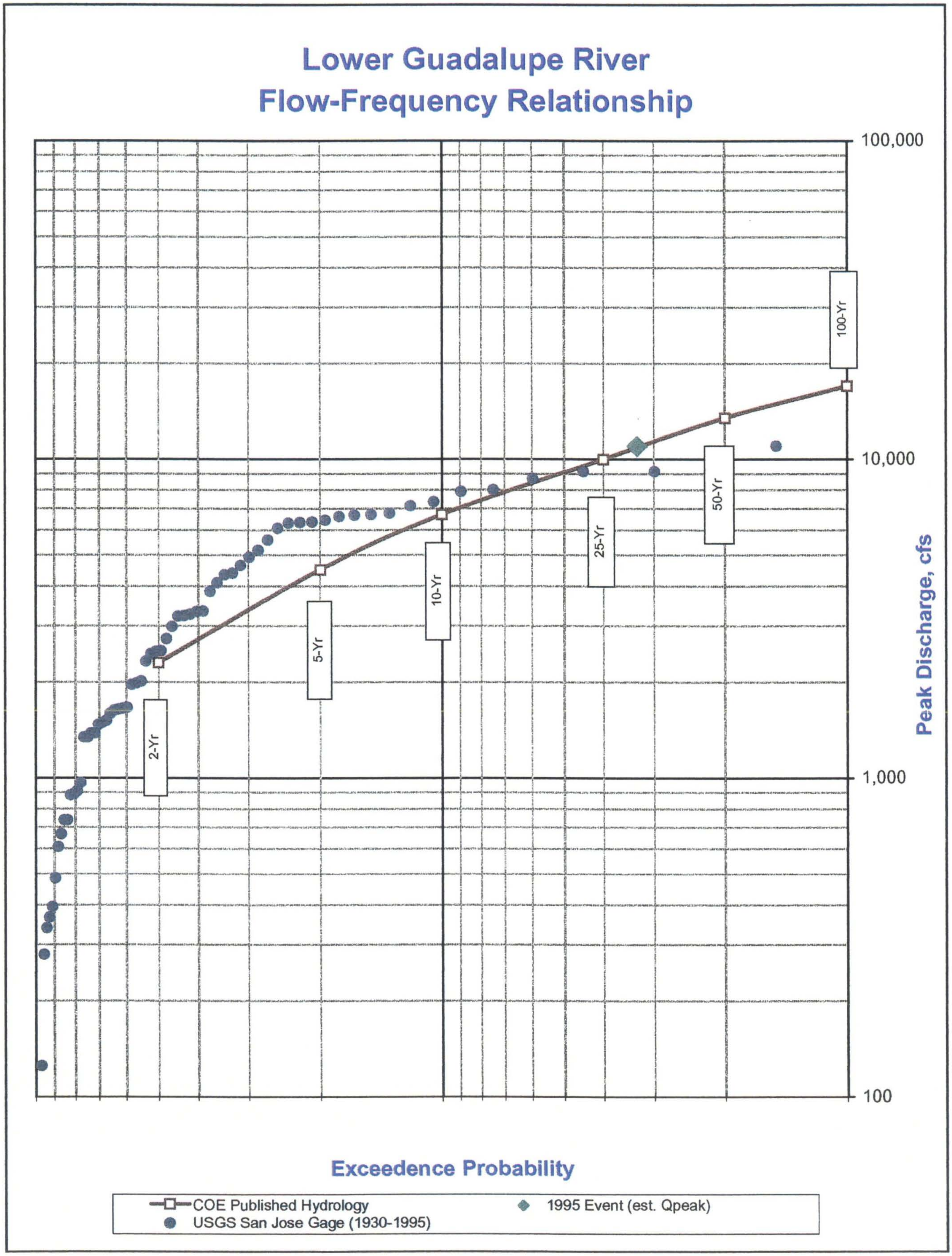
Lower Guadalupe River
Flow-Frequency Relationship
100,000
10,000
1,000
100
Peak Discharge, cfs
Exceedence Probability
2-Yr
5-Yr
10-Yr
25-Yr
50-Yr
100-Yr
COE Published Hydrology
USGS San Jose Gage (1930-1995)
1995 Event (est. Qpeak)

Figure 7.1

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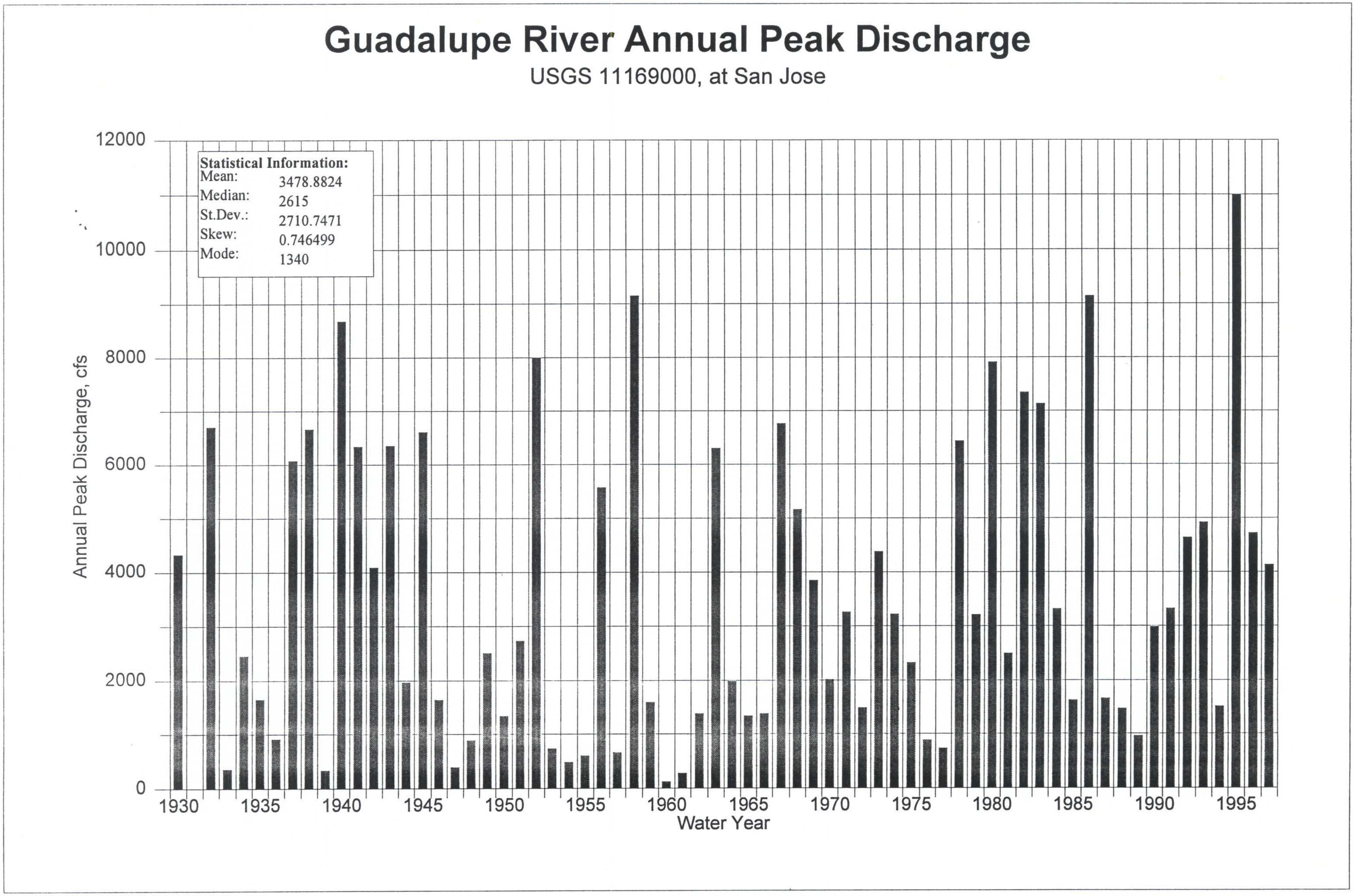

Figure 7.2

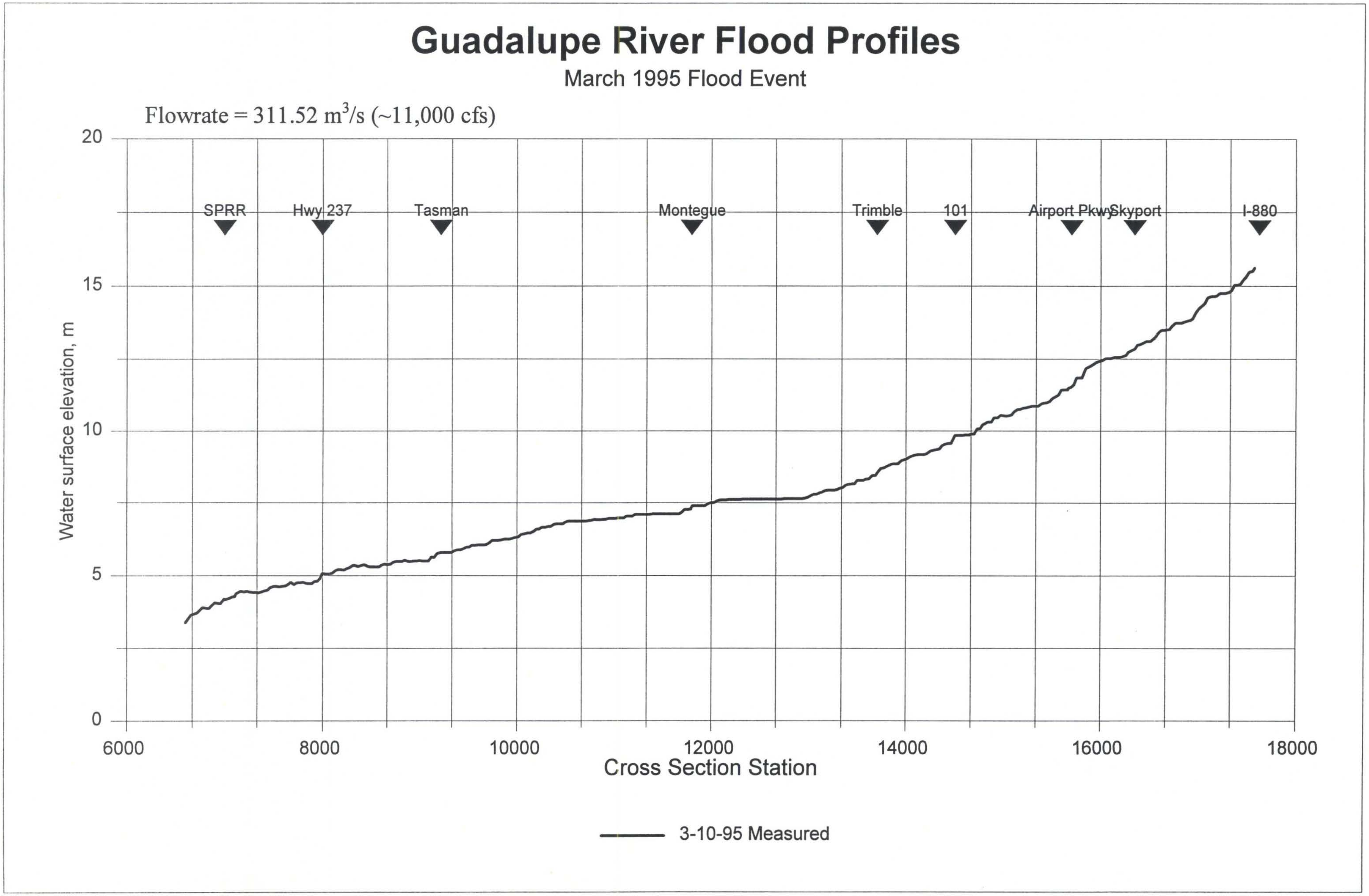
Guadalupe River Flood Profiles
March 1995 Flood Event
Flowrate = 311.52 m³/s (~11,000 cfs)
Water surface elevation, m
20
15
10
5
0
SPRR
Hwy 237
Tasman
Montegue
Trimble
101
Airport Pkwy
Skyport
I-880
6000
8000
10000
12000
14000
16000
18000
Cross Section Station
3-10-95 Measured

Figure 7.3

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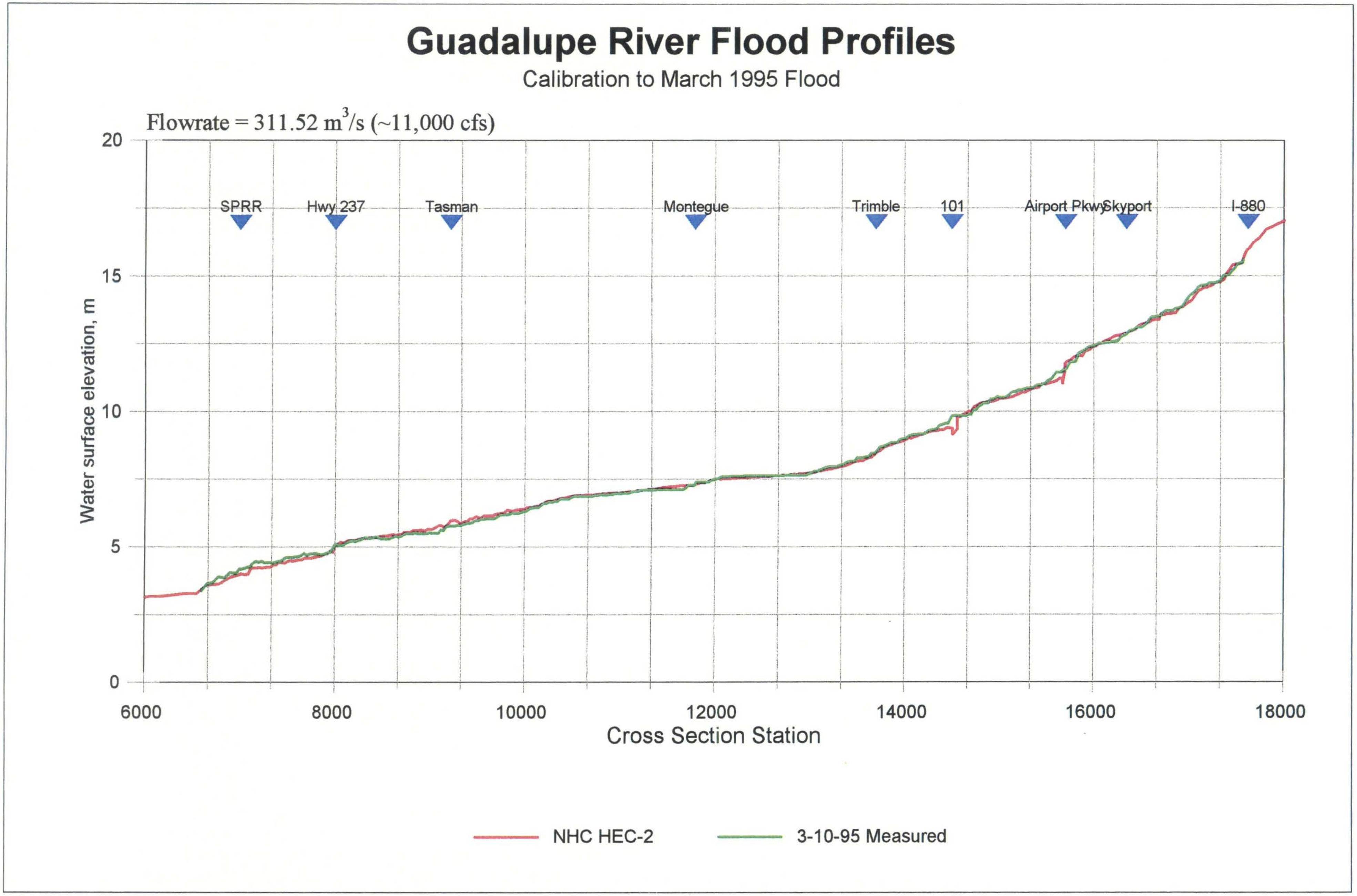

Figure 7.4

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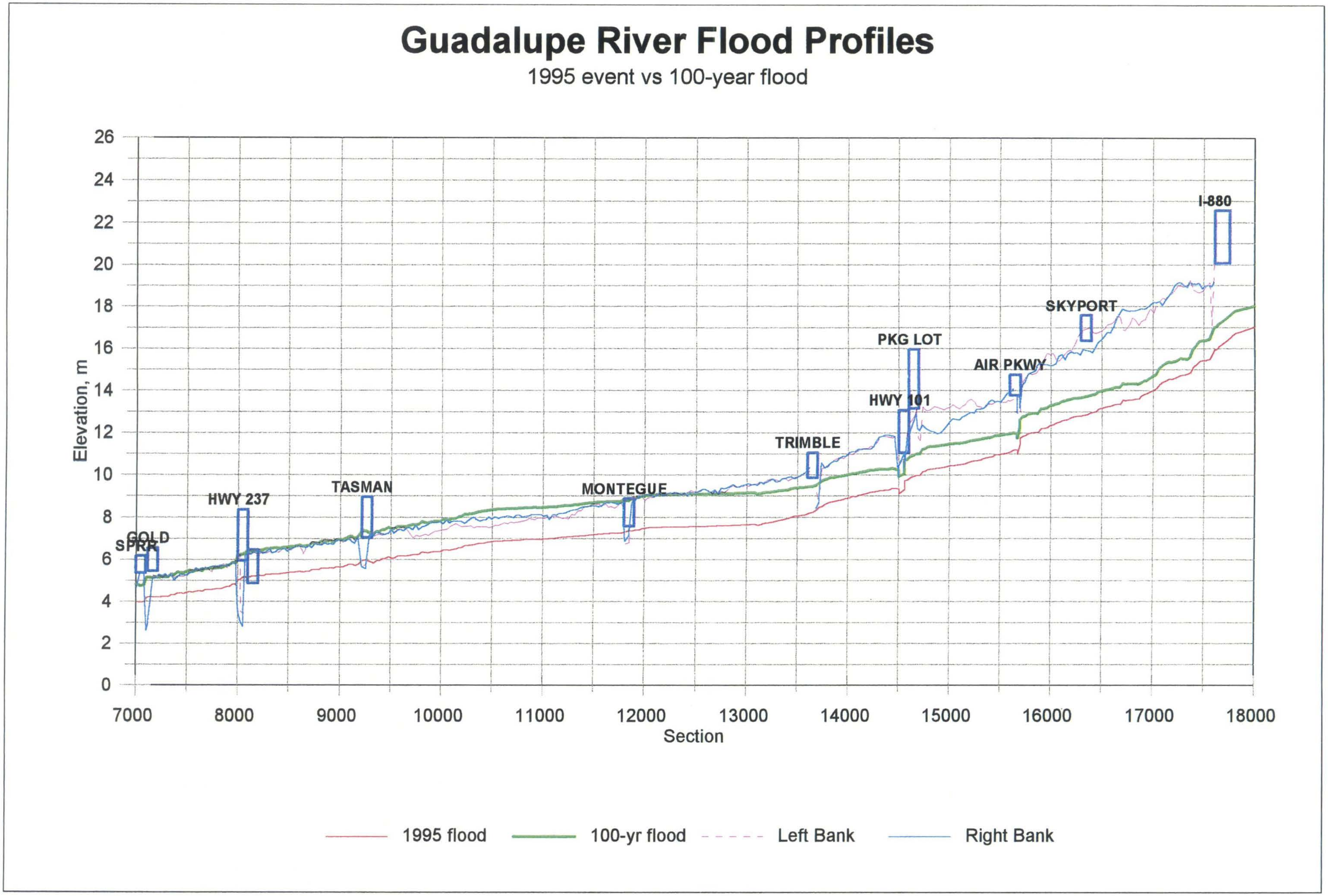

Guadalupe River Flood Profiles

1995 event vs 100-year flood

Figure 7.5

Guadalupe River Flood Profiles

1995 event vs 100-year flood

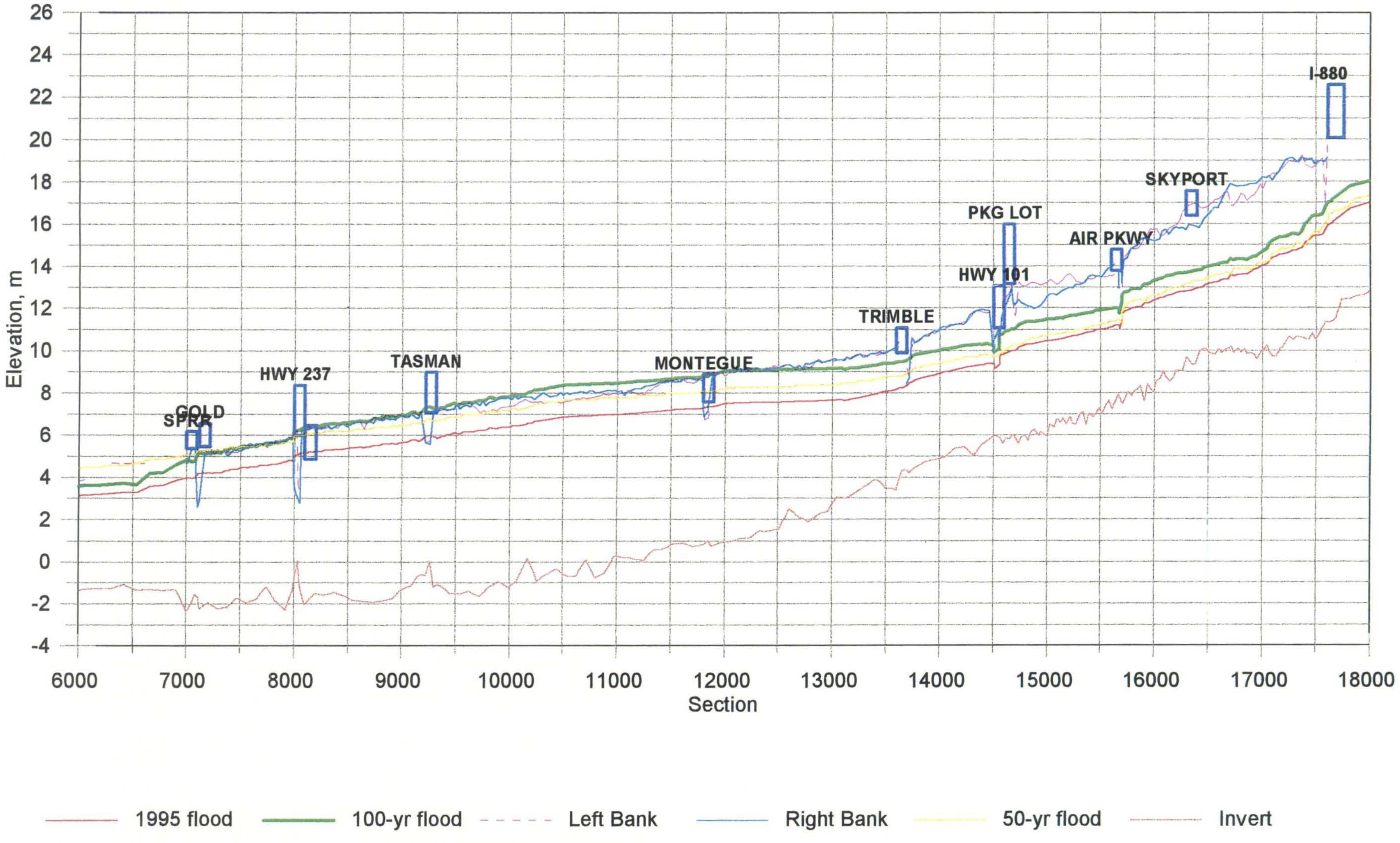

Figure 7.6

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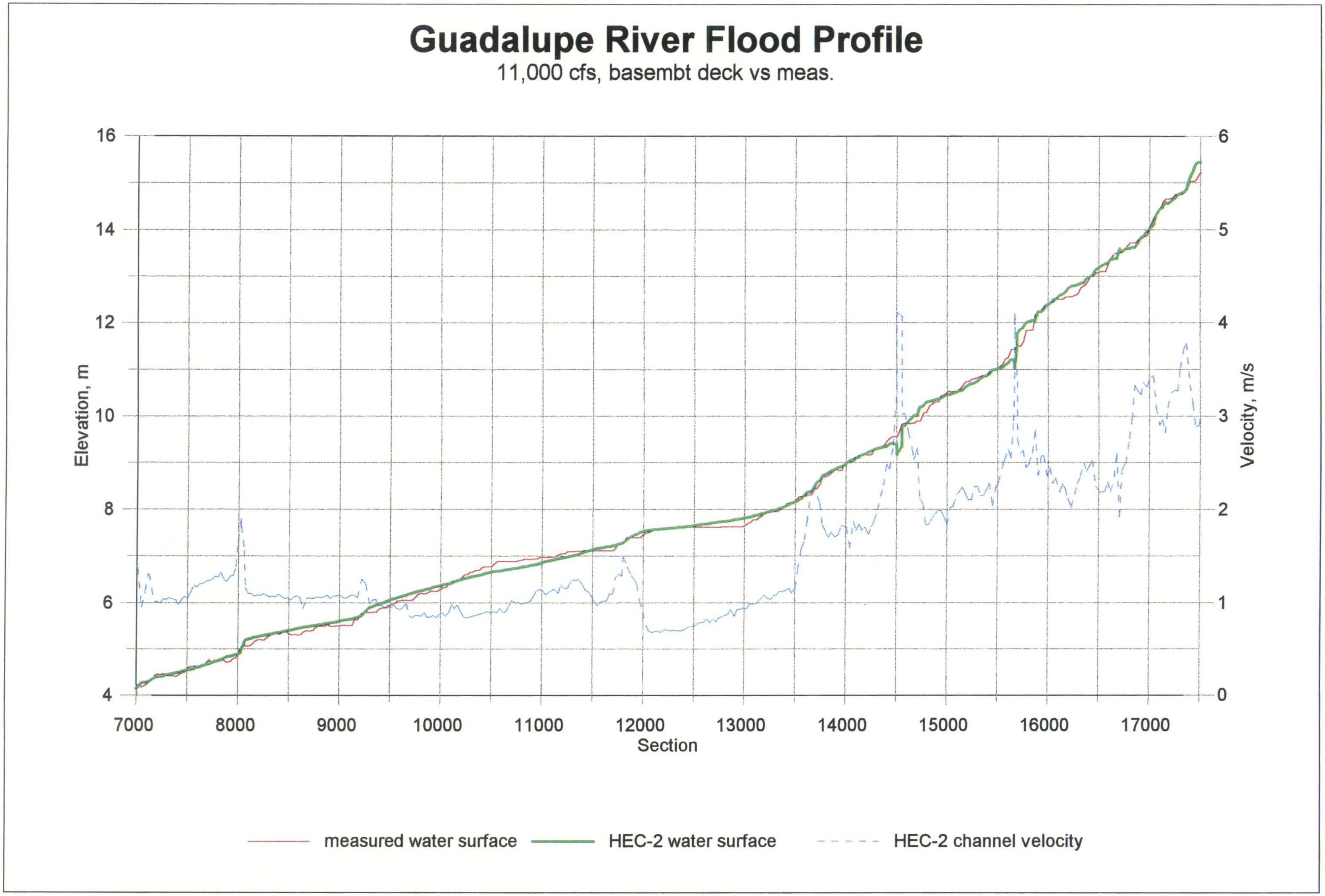
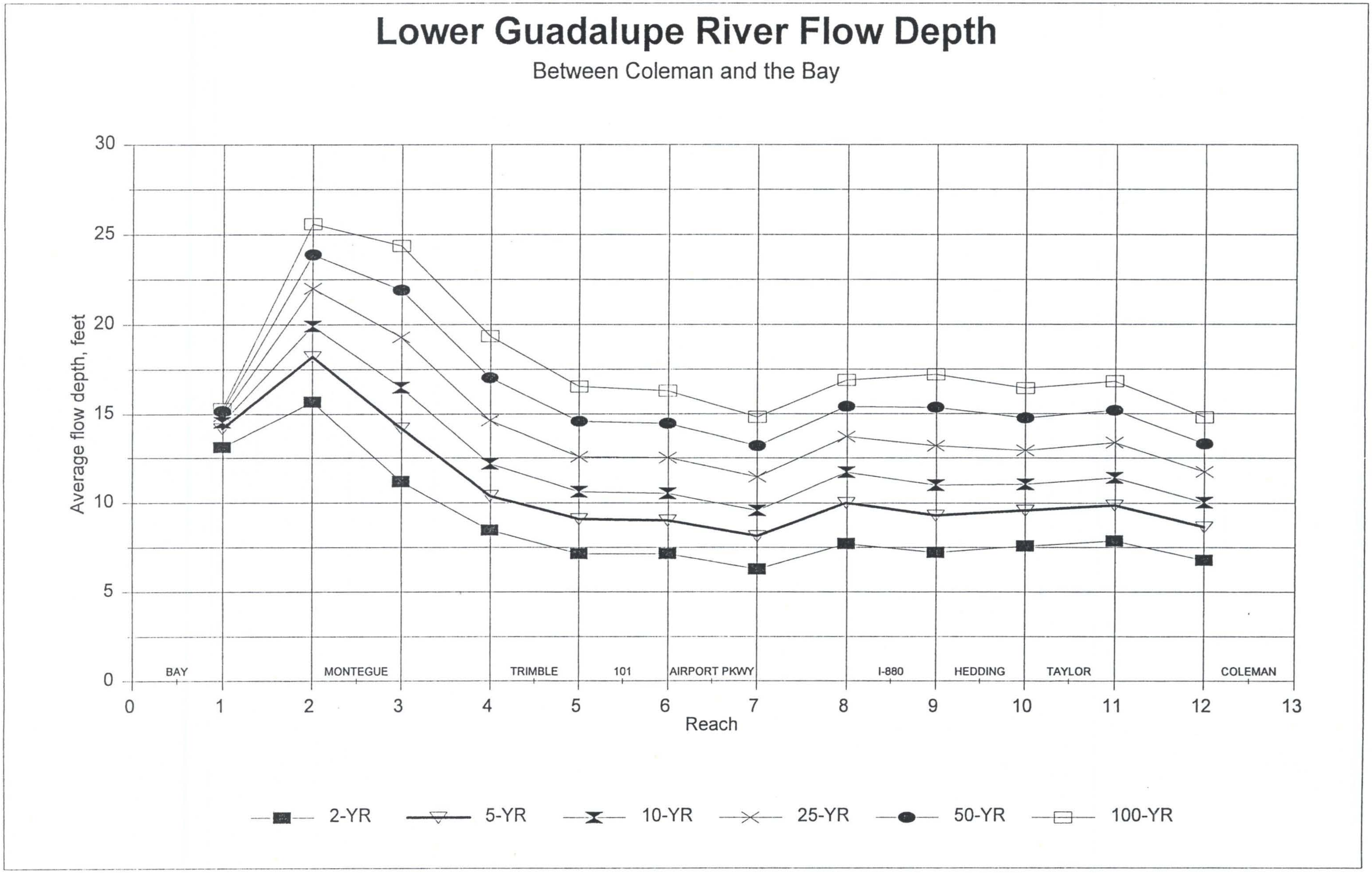


Figure 7.7



Lower Guadalupe River Flow Depth

Between Coleman and the Bay

Figure 7.9

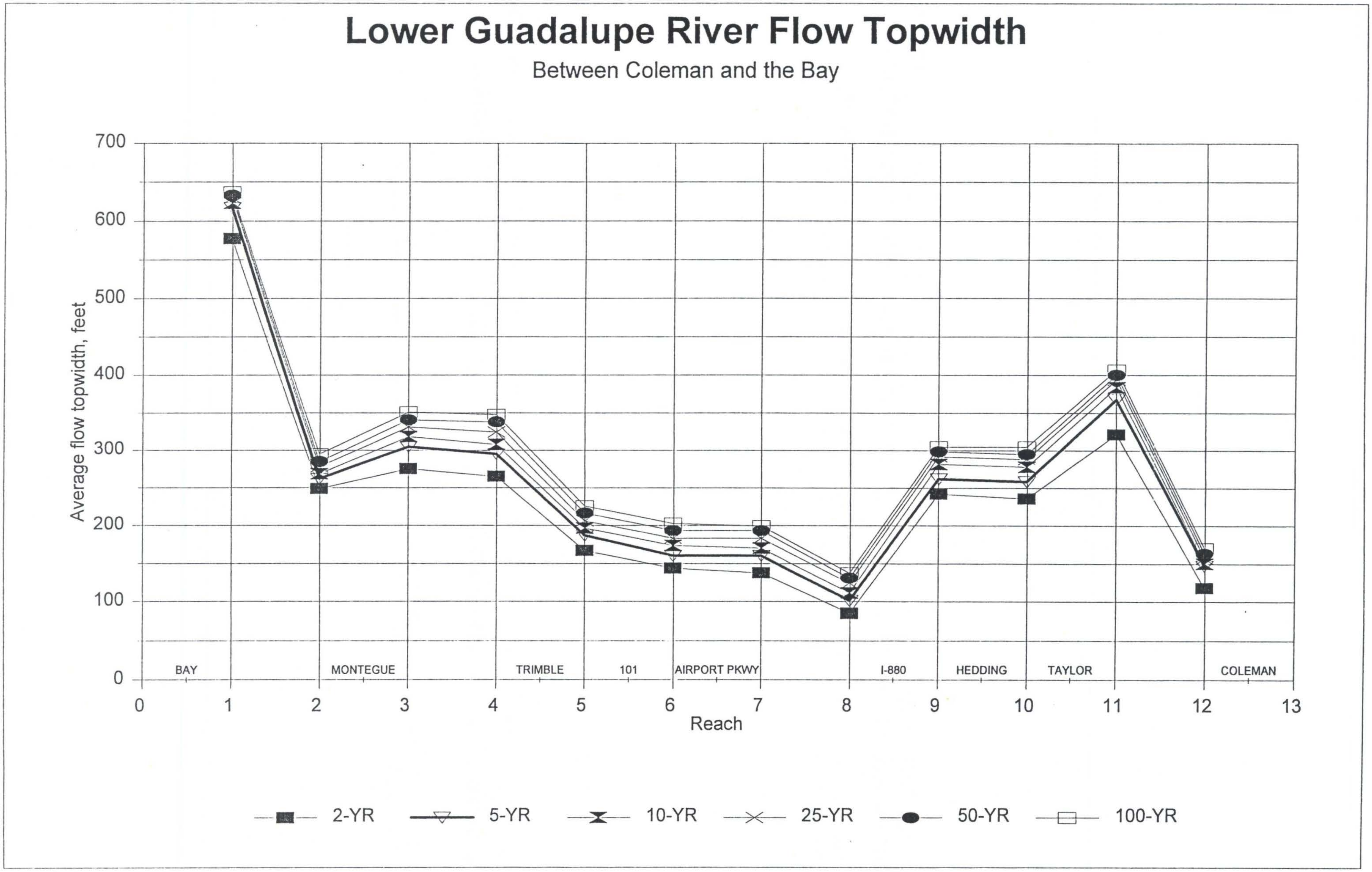

Lower Guadalupe River Flow Topwidth
Between Coleman and the Bay
Average flow topwidth, feet
700
600
500
400
300
200
100
0
BAY
MONTEGUE
TRIMBLE
101
AIRPORT PKWY
I-880
HEDDING
TAYLOR
COLEMAN
0
1
2
3
4
5
6
7
8
9
10
11
12
13
Reach
2-YR
5-YR
10-YR
25-YR
50-YR
100-YR

Figure 7.10

Lower Guadalupe River Flow Velocity

Between Coleman and the Bay

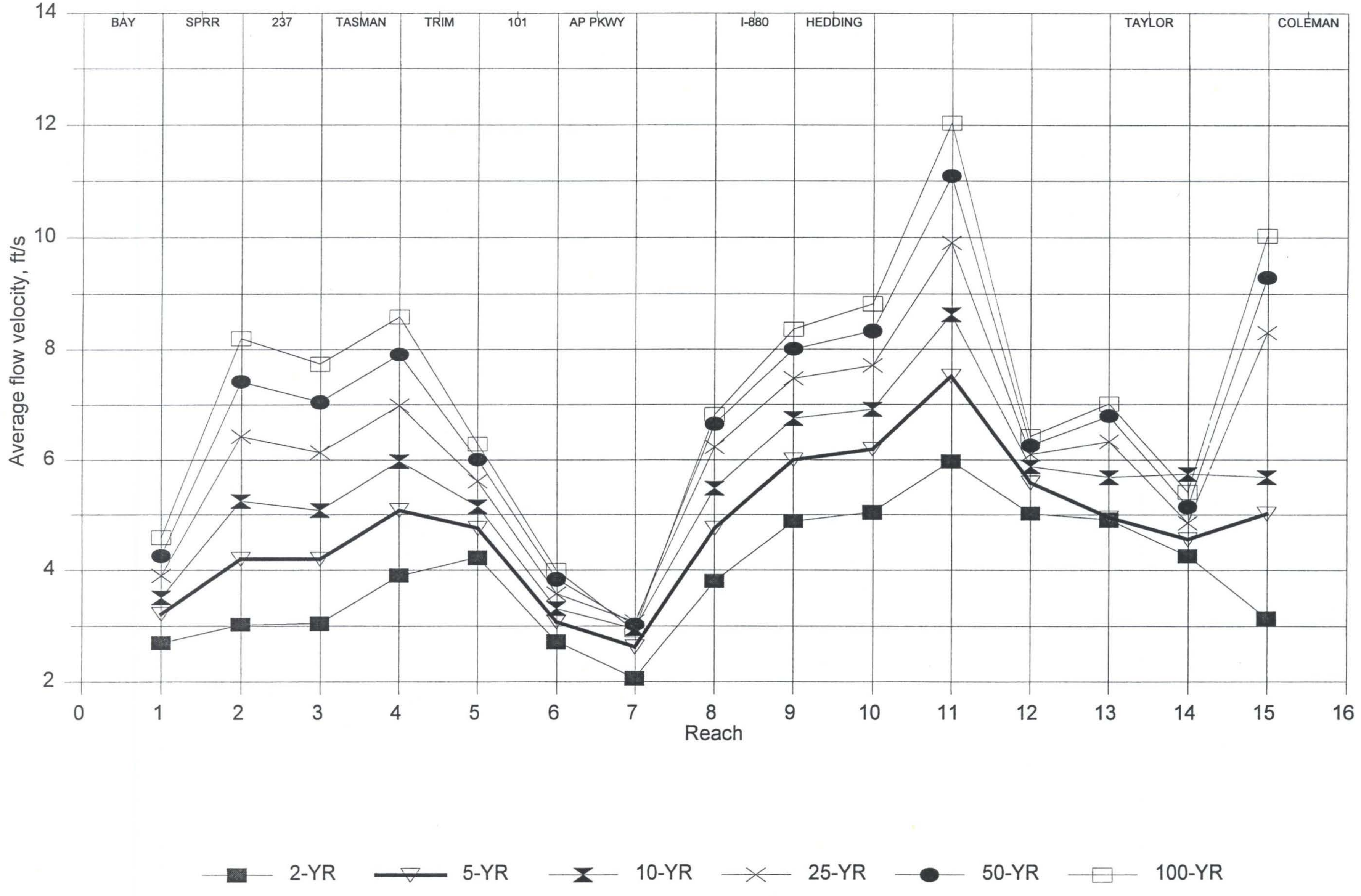


Figure 7.11

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Chapter 8

Sediment Transport Analyses

8. Sediment Transport Analyses

8.1. Purpose of Analyses

The primary purpose of this study is to develop an understanding of the Lower Guadalupe River hydraulics with respect to its ability to convey flow and sediment materials now and in the future. In order to evaluate river hydraulic and sediment transport characteristics of the Project Area historical and field information was gathered and used to develop quantitative engineering tools, including an HEC-6T sediment transport model of the Lower Guadalupe River. These tools are used to evaluate present and future sediment yield to the study reach and to evaluate changing channel capacity and maintenance requirements to meet planning, environmental and design needs of the Santa Clara Valley Water District. Both long term trends (on the order of decades) in river degradation and aggradation as well as episodic single flood events (days) are evaluated for both present (existing) conditions and for two alternative channel designs.

8.2. Approach

Several focused field investigations and sediment sampling expeditions occurred early in the investigation to establish baseline data with respect to riverbed and bank material characteristics throughout the study reach (Chapter 5). Historical information was analyzed to aid in the development of calibration and validation data for hydraulic and sediment transport model development. Measured data, observations of channel performance during low, medium and high flow events, and records of previous channel maintenance requirements are the most reliable indicators of possible future channel performance and change. Key flow, sediment accumulation and grain size data were prepared in appropriate formats for utilization as boundary conditions and for model calibration and production runs.

HEC-2 hydraulic analyses of the river were performed first to establish hydraulic baseline conditions for variations in channel velocity, depth, width, energy slope, and energy losses at the bridges. The Corps' "Sediment Assessment Methodology", or COE-SAM modeling program was then conducted to establish preliminary sediment transport trends and sediment transport potential and sediment loading conditions throughout the Project Reach. Information from the SAM analyses provide data necessary to properly select the most appropriate sediment transport function for materials and flow characteristics found in the Lower Guadalupe River. SAM results also provide an initial estimate of where and how much specific sizes of sediment materials (silts, sands and gravels) collect along the study reach for various flow and upstream sediment loading conditions. Results from the HEC-2 and SAM analyses along with historical information and maintenance records provided the necessary information to develop, test and apply the more comprehensive HEC-6T model to the Project reach. A two-dimensional hydrodynamic model (RMA-2) was developed and applied to the area of the South Bay from Dumbarton Bridge to Alviso and upstream to Highway 880 to develop consistent tidal boundary conditions at the Bay interface. Results from these analyses are presented in Chapter 6. Another RMA-2 model was used to identify lateral flow distributions within the Project Area where lateral flow splits occur between the main channel and an adjacent elevated floodplain. Information from these flow split analyses were utilized in conjunction with the mobile boundary (HEC-6T) modeling to more completely assess hydraulic conditions and sediment transport processes throughout the Project Area.

8.3. Historical Sediment Deposition of the Guadalupe River

Historical sediment deposition along the Guadalupe River was computed using measured cross-sectional data between the Gold Street and Highway 880 bridges. The time period spanning cross-sectional surveys between Gold Street and Airport Parkway was from 1983 to 1996. The time period spanning cross-sectional surveys between Airport Parkway and Highway 880 was from 1987 to 1996.

Sediment deposition was estimated by comparing the 1983-1987 channel volume to the 1996 channel volume. The channel volume was calculated below a reference elevation by applying a standard end area method to the 1983-1987 and 1996 cross section data. The 1983-1987 cross-section data were obtained from the existing conditions HEC-2 input files provided by the Corps and reported in, *Hydraulic analysis of the Guad 104 Reach along the Guadalupe River,* (COE, 1989). The 1983-1987 cross sections were surveyed at approximately a 180-meter spacing. The 1996 data were obtained from the HEC-2 model received from the District. The 1996 cross sections were available at an approximately 120-meter spacing.

The 1983 and 1987 cross section data were based on the vertical mean sea level datum of 1929 (NGVD29). However, the 1996 survey data were based on the NAVD88 vertical datum. The vertical datum conversion of NAVD88 = NGVD29 + 0.81 meters was used to convert the 1983 and 1987 data to the NAVD88 vertical datum used in the 1996 survey.

The estimated sediment deposition in cubic meters per lineal meter of channel length and cumulative volume of sediment deposition downstream of Airport Parkway is shown in Figure 8-1. The results indicate that approximately 176,000 cubic meters of aggradation occurred between Airport Parkway and Gold Street bridges during the time period spanning the cross-sectional surveys. As will be discussed in following sections, these measured amounts of sediment accumulation are used to adjust (calibrate) and validate the sediment transport model - HEC-6T.

8.4. HEC-6T Numerical Modeling

8.4.1. Description of Computer Model HEC-6T

HEC-6T is the result of continued development of the Hydrologic Engineering Center's (HEC) computer program "*Scour and Deposition in Rivers and Reservoirs, HEC-6*" (HEC, 1993). The original HEC-6 code and more recent enhancements to it (HEC-6T) were planned, designed, coded and enhanced by William A. Thomas. The original version of HEC-6 was developed and applied in 1967 by Thomas to calculate the location and volume of sediment deposition in the Ozark Reservoir for the Arkansas District Corps of Engineers. Since then the model has been greatly enhanced by Thomas, WES and HEC to become a world-wide industry standard for one-dimensional sediment transport routing and analysis. Sediment routing analyses conducted for the Lower Guadalupe River Investigation were performed using the most recently advanced version of the HEC-6 model, referred to as HEC-6T (Thomas, 1996). Following are brief descriptions of the basis, assumptions, data requirements, capabilities and limitations of this model.

8.4.2. Basis and Capabilities

Computer program HEC-6, "Scour and Deposition in Rivers and Reservoirs" is a one-dimensional, movable boundary, open channel flow model designed to simulate streambed profile changes resulting from varying river flow and reservoir conditions. The model is based on one-dimensional, gradually varied flow hydraulics and sediment transport theory. The model is capable of calculating sedimentation in dendritic, closed loop and distributary networks of river systems. The geometry is modeled by cross sections and the hydraulic roughness is either assigned by Manning n-values or calculated by bed roughness predictors. Networks can contain up to 24 segments and a total of one thousand cross sections. Up to 500 elevation-station points are permitted per cross section. Up to ten local inflow and outflow points can be modeled on each segment of a network, simultaneously. A continuous sequence of flows (a flow hydrograph) is segmented into a series of steady flow events of variable duration.

For each flow sequence, the conservation of energy equation (Equation 1, below) is solved to determine the water surface profile and pertinent hydraulic parameters such as energy slope, velocity, depth, hydraulic roughness and width at each cross section. Sedimentation processes, i.e. the erosion, entrainment, transportation, deposition and the compaction of sediment particles, are then computed at each cross section, by solving the sediment continuity equation (Equation 2, below) and a sediment transport function of the form of Equation 3 below.

Various supporting equations which model the bed sorting processes, the armoring processes, particle entrainment, particle deposition and consolidation of mixtures of bed deposits are required. The basis for this solution of the non-equilibrium condition of sediment transportation is Thomas's extension of the Einstein model of the equilibrium condition.

Conservation of Energy Equation 1:

$$\frac{\partial H}{\partial X} + \frac{\partial\left(\alpha \frac{V^2}{2g}\right)}{\partial X} = S$$

where H = water-surface elevation, ft
X = distance in the direction of flow, ft
α = coefficient for the horizontal distribution of velocity
V = average flow velocity, ft/sec
g = acceleration due to gravity, ft/sec^2
S = slope of energy grade line

Continuity of Sediment Material Equation 2:

$$\frac{\partial G}{\partial X} + B\frac{\partial y_s}{\partial t} = q_s$$

where G = rate of sediment movement, ft^3/day
B = width of movable bed, ft
y_s = change in bed surface elevation, ft
t = time, days
q_s = lateral inflow of sediment, ft^3/ft/day

Sediment Transport Function Equation 3: $G = f(V,y,B,S,T,d_{eff},d_{si},P_i)$

where y = effective depth of flow, ft
T = water temperature, F
d_{eff} = effective grain size of sediment in size class i
d_{si} = geometric mean of class interval
P_i = percentage of ith size class in the bed

8.4.3. Data Requirements and Program Capabilities

Data requirements include: (1) geometry information (in typical HEC-2 format), including channel shape, cross section spacing, bed slope, and bed roughness, (2) hydraulic information, including a continuous sequence of flows and durations, water temperature and specification of hydraulic controls and/or boundary conditions, and (3) sediment information, including bed material characteristics, inflowing load quantity and grain size distribution, bed armoring characteristics and selection of the sediment transport function or functions to be used for simulating sediment movement (scour and deposition).

General program capabilities are listed below. More detailed explanations of the theory and methods used for computing sediment transport quantities are found in the new users manual (HEC, 1993).

8.4.4. Assumptions

The following assumptions are made in deriving and applying these equations:

a. The channel is sufficiently straight and uniform in the reach so that the flow characteristics may be physically represented by a one-dimensional model.

b. The velocity is uniformly distributed over the cross section.

c. Hydrostatic pressure prevails at every point in the channel.

d. The water surface slope is small.

e. The density of the sediment-laden water is constant over the cross section.

f. The unsteady flow resistance coefficient is assumed to be the same as for steady flow in alluvial channels and is approximated from resistance equations applicable to alluvial channels or from field surveys.

The numerical technique used to solve Equation 1 is commonly called the standard step method (HEC, 1985; Henderson, 1966; French 1985). An explicit six-point finite difference method is used to solve Equation 2 as functions of time and sediment transport rates. These rates, combined with the duration of the flow, allow for volumetric accounting of sediment within each reach. The amount of scour or deposition within each reach is computed and the cross section geometry is then adjusted accordingly. The computations proceed to the next flow in the sequence and the cycle is repeated beginning with the updated geometry. The sediment calculations are done by grain size fraction, thereby allowing for the simulation of hydraulic sorting and armoring (although the details of these processes are not well known). HEC-6T can also be executed in a fixed-bed mode, similar to HEC-2, in which only steady water surface profiles are computed with no consideration of sediment transport effects in the channel.

Potential sediment transport characteristics and rates (including sediment load, gradation of the load, gradation of the bed surface materials and depth of scour and or deposition) are computed at each cross section via the sediment continuity equation (Eq-2) and user-selected transport functions of the form shown in Eq-3.

Users can select from the following nineteen (19) sediment transport functions:

Bed Material Load

1. Toffeleti's (1969) Relationship
2. User specified curves for transport based on observed data
3. Madden's (1963) modification (unpublished) of Laursen's (1958)
4. Relationship
5. Yang's Stream power for Sands Relation (1972)
6. Duboys (Brown, 1950)
7. Einstein Total Load (Einstein, 1950)
8. Ackers-White (1972)
9. Colby (1964)
10. A combination of Toffaleti (1969) and Schoklitsch (1930)
11. Meyer-Peter and Muller (1948)
12. Schoklitsch (1930)
13. A combination of Toffaleti (1969) and Meyer-Peter and Muller (1948)
14. Madden's (1985) modification of Laursen's (1958) relationship
15. Laursen-Copeland (1989)
16. Parker
17. Profitt-Sutherland
18. Brownlie (1981)
19. Yang's Hyper-concentration Equation (1996)

Cohesive Sediment Transport

A method has been assembled from the work of Partheniades, (1965), Ariathurai (1976) and Krone (1962) Methods.

8.4.5. Limitations

HEC-6T has the following limitations:

1. HEC-6T is based on one-dimensional gradually varied flow hydraulics and sediment transport theory.

2. There is no provision for simulating the development of channel meanders or specifying lateral distributions of sediment load across a cross section.

3. The processes and loading contributions due to bank caving are calculated by empirical methods rather than by the physics of soil mechanics.

8.4.6. Boundary Conditions

As described in the previous section, HEC-6T is a numerical model that computes and simulates: (a) the hydraulics of gradually varied open channel flow, (b) conservation of mass of sediment transport by updating the channel bed elevation and bed material grain size distribution over time in response to the inflowing sediment load and local sediment transport capacity and, (c) allows for the coupling of the resultant changes in channel longitudinal profile and cross-sectional changes with the hydraulic characteristics of the channel. To accomplish these computations, boundary conditions are specified at inflow and outflow control points that bracket the study area. HEC-6T requires that flow rate, sediment concentration and grain size distribution, and the controlling water surface elevation at the downstream limit of the model be specified for the selected time period for simulation. In addition, internal boundary conditions that define the cross-sectional geometry, reach lengths between cross-sections, hydraulic roughness, bed material grain size distribution, and limits of sediment deposition and erosion at specific cross-sections are specified. For this application of HEC-6T to the Lower Guadalupe River system the following data sources were used to specify boundary conditions:

1. Geometry - the 1996 District cross-section survey of the Guadalupe River provided the geometric data base for the HEC-6T model

2. Flow rates - Flows measured by the USGS at the Guadalupe River gage near San Jose (gage no. 11169000) were used as the inflow boundary condition for the HEC-6T model. Design hydrographs and peak discharges for specific frequency flood events were derived from the Corps of Engineers' (1977) Hydrology Report for the Guadalupe River.

3. Sediment Concentration - Sediment concentration was measured periodically at the Guadalupe River near the San Jose gage by the USGS. A sediment discharge rating curve was developed from these data and used as the sediment inflow boundary condition for the HEC-6T model

4. Starting Water Surface Elevation - The HEC-6T model extends to the confluence of the Guadalupe River with San Francisco Bay. Tidal stage records measured at Dumbarton Bridge in south San Francisco Bay and were extended to the downstream limit of the HEC-6T model with the two-dimensional RMA-2 model of south San Francisco Bay. These tidal stage elevations were used to develop starting water surface elevation conditions for the downstream end of the study reach

5. Bed Material Grain Size Distribution - Bed materials were sampled throughout the study reach and used to characterize the longitudinal variation of grain size distribution in the study reach.

6. Calibration and model adjustment data - HEC-6T was calibrated using data from the 1995 flood event, comparison of sediment accumulation rates derived from analysis of historical cross sections and District channel maintenance records.

8.4.7. Model Adjustment

There are two primary phases of adjustment of the HEC-6T model. These phases are the hydraulic model adjustment and sediment transport model adjustment. Hydraulic model adjustment primarily consists of adjusting effective conveyance limits, Manning's n values, and expansion and contraction energy loss coefficients to obtain reasonable agreement between measured and computed hydraulic information. By providing a measure of the channel's stage-discharge characteristics and the frictional energy loss, measured water surface profiles for a known discharge provide adequate information for hydraulic model adjustment. The HEC-2 model and measured high water profile data from the 1995 flood event described in Section 7 served as a basis for HEC-6T hydraulic model adjustment. A uniform composite n value is a satisfactory way to specify hydraulic roughness for a single discharge and flow stage. However, since bed material sediment transport occurs for all discharges greater than the discharge required to initiate bed mobility, Manning's n values were specified separately for the channel and overbank sections. The resultant composite n value thus varies with the discharge and resultant flow stage. In addition, Manning's n which decreases as flow stage increases was specified to account for the fact that the relative roughness of flexible vegetation decreases with increasing flow submergence and velocity (Lopez and Garcia, 1998). Using previously adjusted HEC-2 model developed by NHC as a starting condition, slight modifications to the Manning's n values and expansion and contraction energy loss coefficients yielded reasonable agreement with the measured high water profile for the 1995 flood event. Manning's n values adopted for the channel portions of each cross-section varied between 0.030 to 0.028 for the entire modeled reach. Overbank n values varied from a high of 0.200 for conditions with thick vegetation and/or emergent bulrush and tules (Hall and Freeman, 1994) to 0.050 for highly submerged brush and grass. The resultant water surface profile computed by HEC-6T is shown on Figure 8-2 and compared with the measured 1995 flood profile as well as the water surface profile computed by HEC-2 for this event.

Measurements of sediment concentration and flow rate were completed by the USGS at the Guadalupe River at San Jose gage during the time period from 1957 to 1962. To our knowledge, these are the only reliable measurements of sediment discharge made in the vicinity of the project reach. To increase the reliability of the HEC-6T model for predicting sedimentation processes in the project reach, the HEC-6T model was extended upstream to the location of the stream gage. This allows a rational application of the measured sediment discharge relationship at this location as a boundary condition for sediment transport computations in the project reach. Extending the model upstream to this location encompasses several off channel conveyance areas located between the Coleman and Hedding Street bridges. These conveyance zones necessitated the development of looped channel simulation capabilities within the HEC-6T source code. These capabilities greatly increase the abilities of HEC-6T to simulate sedimentation in bifurcated (flow split) channel systems as exists in this portion of the Guadalupe River.

In addition to total suspended sediment concentration, the USGS analyzed the samples for total suspended sand (grain diameter > 0.0625 mm) and total suspended silts and clays (grain diameter < 0.0625 mm). The measured sediment concentrations, percent suspended sand, and flow rate are provided in Table 8-1. A sediment discharge rating curve was developed from this information, and the grain size distribution of the inflowing sediment load was developed from and application of the SAM hydraulic design package using local hydraulic conditions computed at the USGS gage and measured bed material grain size distributions. The bed material at, and upstream of this location includes negligible amounts of medium sand (0.5 mm > grain diameter >0.25 mm), fine sand (0.25 mm > grain diameter > 0.125 mm), or very fine sand (0.125 mm > grain diameter > 0.0625 mm). These sand sized sediments, however, are transported past this location as washload and are not present in the bed materials in this reach. Direct determination of the quantity of wash load can not be determined without an independent measurement (monitoring) of sedimentation dynamics in the study reach.

Table 8-1. Measured Suspended Sediment Discharge Guadalupe River, Gage No. 11169000

Flowrate, cfs	Total Suspended Sediment Discharge, Tons/day	Percent Finer than 0.0625 mm	Total Suspended Sand Discharge, Tons/day
13	70	86	9.8
84	130	77	29.9
620	4150	61	1618.5
587	1500	72	420
548	1370	68	438.4
344	947	61	369.33
272	1620	92	129.6
97	149	94	8.94
1450	16200	68	5184
675	10900	69	3379
465	1260	86	176.4
481	1870	84	299.2
88	113	74	29.38
77	761	85	114.15

Measured changes in historical cross-sectional geometry provides an independent means to assess the relative quantity of wash load passing the upstream boundary of the study reach. By computing an accurate balance of the sediment storage dynamics in the study reach, the relative quantity of fine wash load entering the reach can be determined by adjusting the fine sediment inflow until a reasonable agreement between computed and measured sediment aggradation and grain size is achieved. The resultant measured and computed total bed material sediment load, and total (bed material and wash load) inflowing sediment load is shown on Figure 8-3. Measured sediment accumulation for the same time period in which the cross-sectional surveys were compared to the resultant sediment deposition computed using the HEC-6T model for this time period are shown in Figure 8-4. The results indicate deposition of the gravel and coarse sand load in the subreach bounded by Airport Parkway and Trimble Road. Significant deposition of medium and fine sands occurs in the reach bounded by Trimble Road to a point near the upstream limit of tidal influence at Tasman Avenue. Downstream from this point channel aggradation is caused by a combination of tidal marsh accretion (Chapter 6) and deposition of fine watershed derived wash load components during discharges lower than that required to mobilize the bed at the upstream end of the study reach. Combining the HEC-6T long term tidal simulation with the HEC-6T bed mobilizing single event flood simulation provides an estimate of the total watershed derived sediment deposition within the study reach. The resultant unit sediment deposition by grain size classification is shown in Figure 8-4. The net accumulated sediment load for existing flood conditions and long term average annual flow conditions subject to tidal fluctuations are shown in Figures 8-5 and 8-6, respectively.

Sensitivity of the project reach channel response to variation in the inflowing sediment discharge rating curve is shown in Figure 8-7. A twenty percent increase in sediment concentration was assumed, and the resultant sediment deposition pattern computed. The results indicate a proportionate increase in sediment deposition in the project reach, but the locations of sediment deposition are independent of the magnitude of inflowing sediment load. Thus, if changes to the sediment loading of the Guadalupe River occurs in the future due to fires, land use changes, landslides, or other means, then channel aggradation will continue to occur primarily at the locations where aggradation is presently identified. The magnitude of channel aggradation will change as a function of the magnitude of the inflowing sediment load and gradation. If channel maintenance activities are envisioned as a means of mitigating for channel aggradation, then regular channel maintenance will be required in the future for as long as sediment flows into the study reach.

The following sections describe the application of the adjusted HEC-6T model to evaluate existing sedimentation dynamics along the lower Guadalupe River, and identify potential channel maintenance alternatives for sedimentation and flood management.

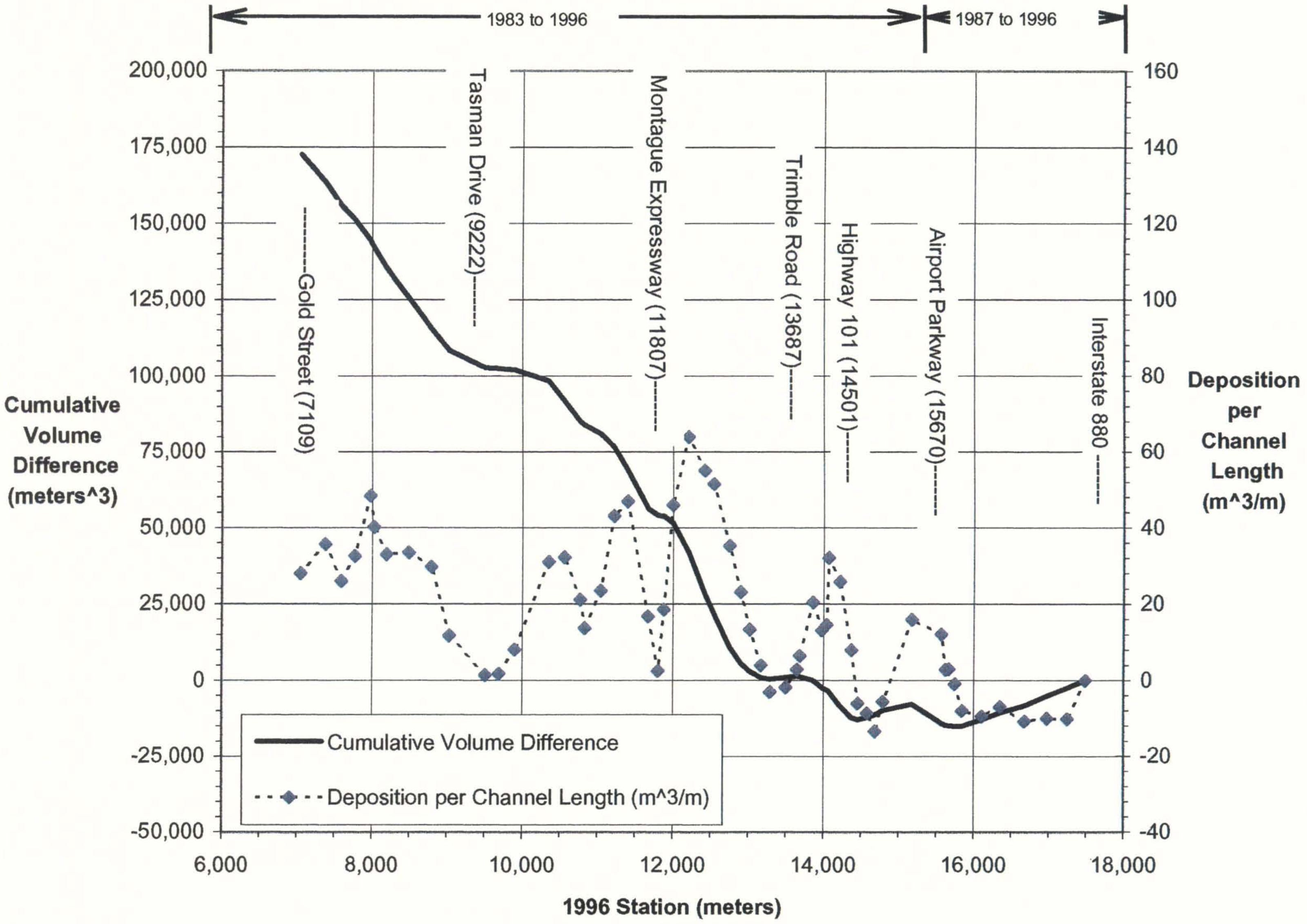


Figure 8-1. Cumulative Volume Difference between 1983/1987 and 1996 Cross Sections

Figure 8-2. Flood Profile Comparison

1995 flood discharge

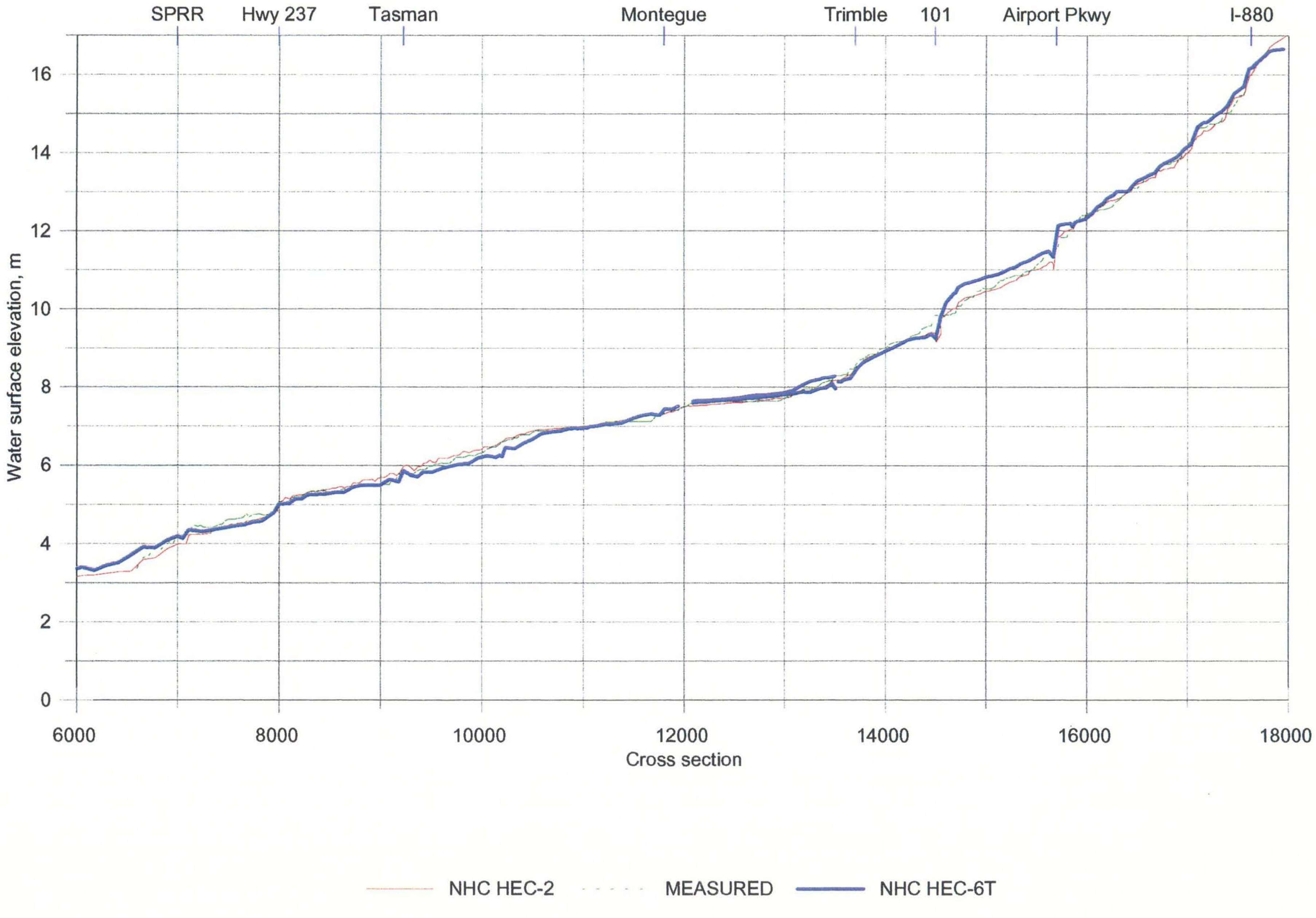
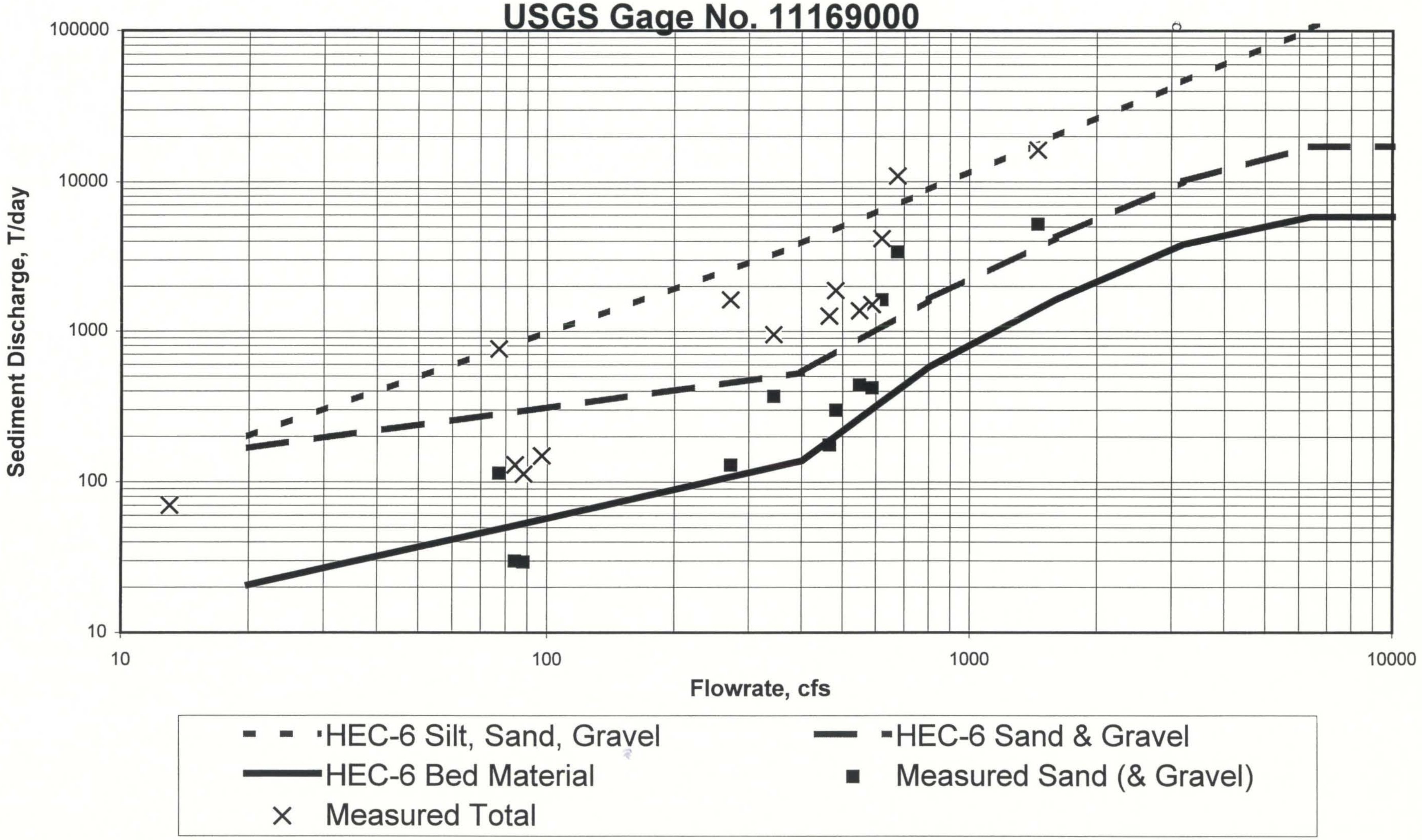


Figure 8-3. Sediment Discharge Rating Curves
Guadalupe River U/S I-880
USGS Gage No. 11169000



rating

Figure 8-4 Flood Condition Load Calibration

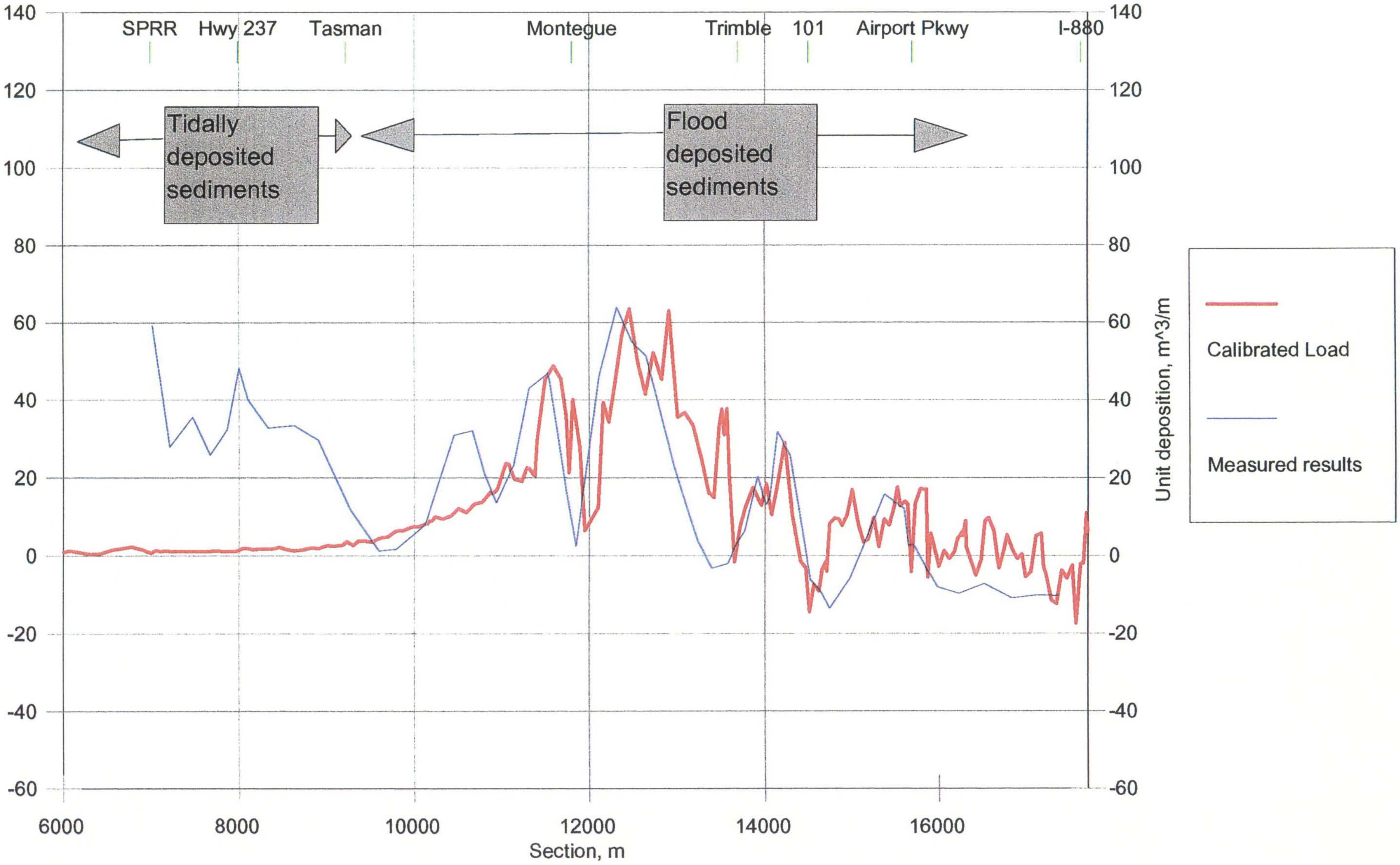


Figure 8-5 Flood Conditions, 1983-1995

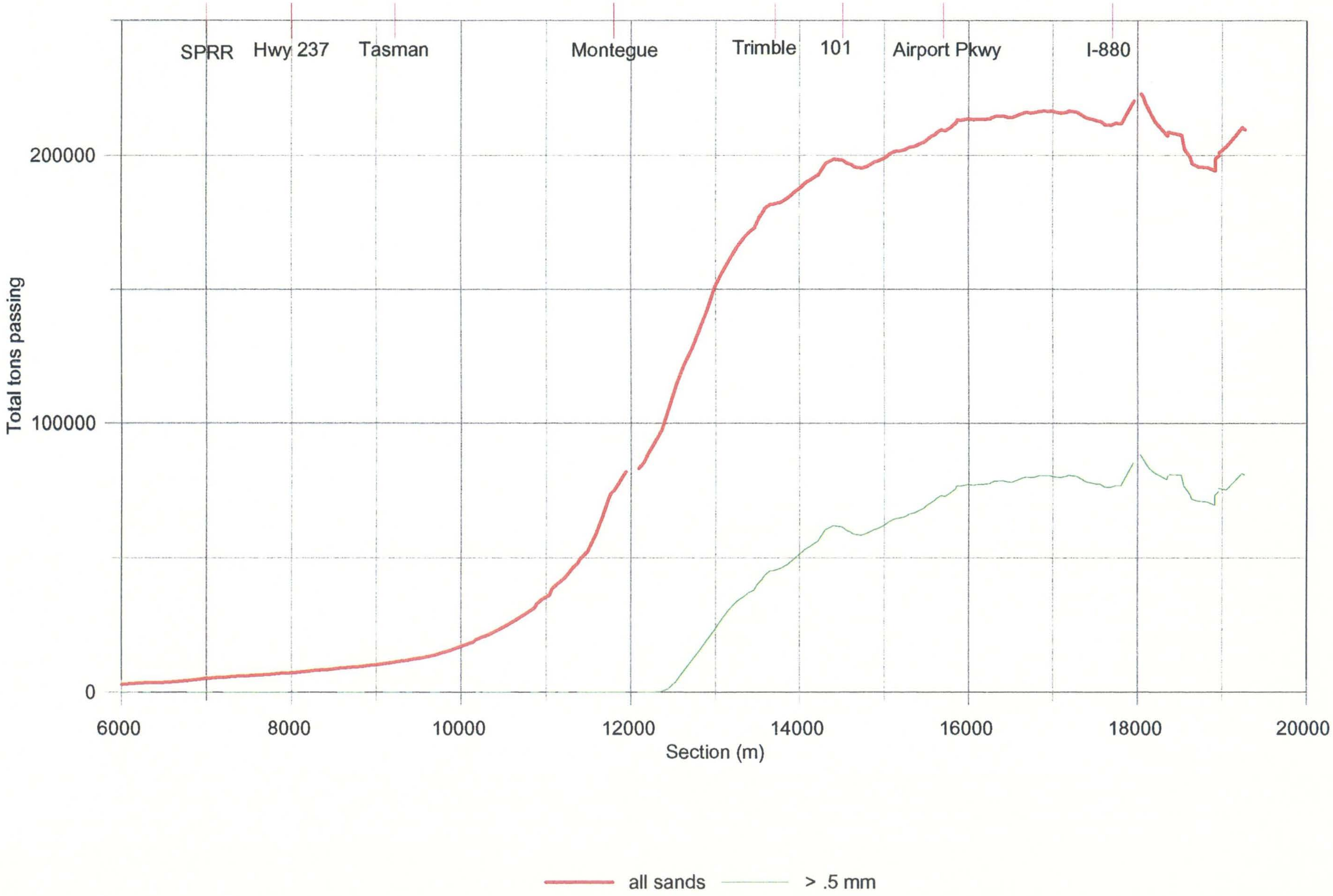


Figure 8-6. Accumulated Sediment Load

12 year tidal condition simulation

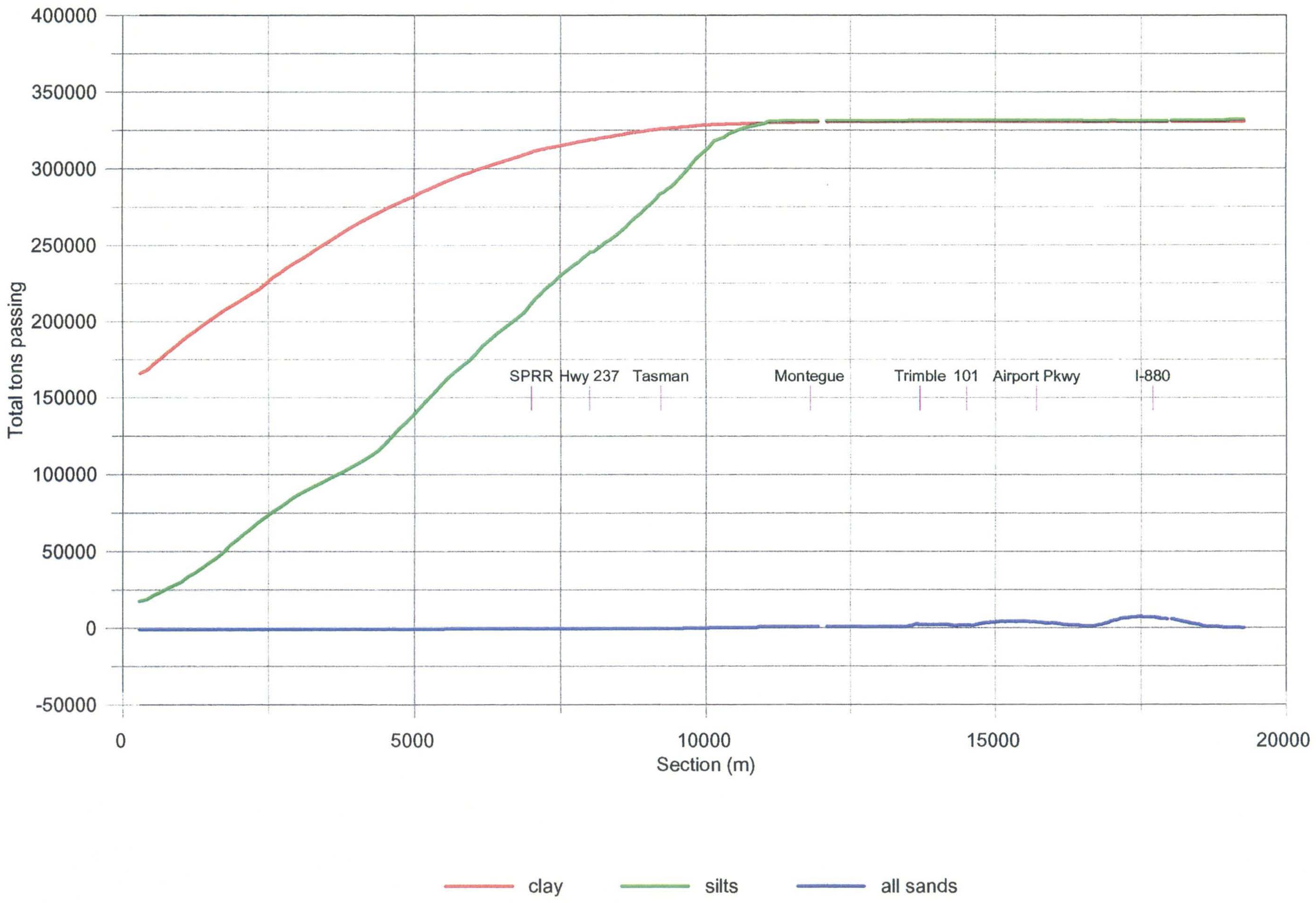
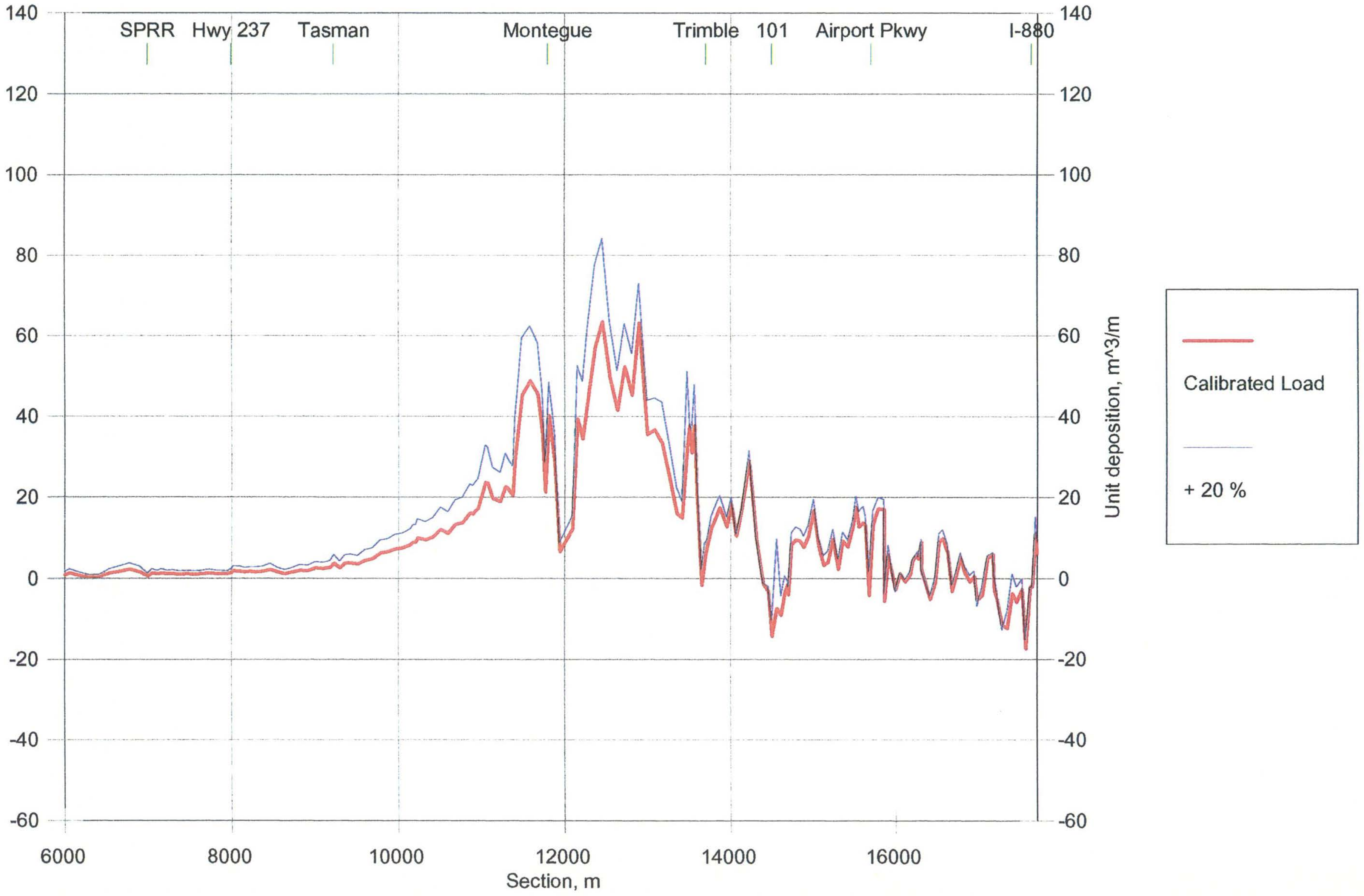


Figure 8-7. Inflow Sediment Load Sensitivity



Sedimentation Analysis of Existing Conditions

Chapter 9

9. Sedimentation Analysis of Existing Conditions

Of primary concern for the District is the ability of the existing channel improvements to pass the 100-year flood now and in the future. Application of the adjusted HEC-6T model allows for the computation of single storm event and long term sediment trends, storage dynamics and changes in channel hydraulics.

Long Term Trends - Long Term (i.e. > 50 years) changes in channel geometry and flood profiles were computed. The hydrologic conditions used for long term simulations were developed by sequentially repeating the observed 1983-1995 hydrology used for sediment model calibration. The observed hydrologic pattern was repeated 6 times for a total simulation period of 72 years. Changes in channel capacity over the simulation period due to sediment storage dynamics are conveniently visualized by plotting *long term changes* in the computed 100-year (0.01 probability of exceedence) *water surface elevation at key locations*. These results are shown on Figures 9-1 and 9-2. Without regular sediment maintenance, the long term accumulation of watershed derived sediments in the reach from Hwy 101 to Montague (Figure 9.1) may increase the 100-year food stage from 5 to 7.5 feet (1.5 to 2.3 m). As is exemplified by the measured sediment accumulations in the reach bounded by Airport Parkway and Tasman Avenue, the largest stage changes in the river occur in the vicinity of greatest sediment deposition. This portion of the Guadalupe River is a natural location for sediment accumulation of flood derived watershed sediments due to the reduction in channel slope and flow velocity in this vicinity. Channel capacity of the project reach will decrease over time as channel aggradation progresses. Similarly, long term average annual discharge (~ 30 cfs) simulations subject to tidal stage fluctuations were performed. Sediment aggradation of primarily wash load materials may contribute to approximately 0.25 to 0.5 ft (0.076 to 0.15 m) increase in the 100-year flood stage in the lower portions of the study reach in the vicinity of the SPRR and Highway 237 bridges as shown in Figure 9-3.

Study results indicate that most of the project reach, especially that reach between Trimble and Montague, is located in a zone of active channel aggradation. The aggrading sediments are derived from both watershed and tidal sources. The gradation of the aggrading sediments change from gravels and coarse sands in the vicinity of Airport Parkway, and grade down to medium to fine sands from Trimble to Tasman Avenues. Downstream of Tasman Avenue, silt and clays from watershed sources are deposited under low flow conditions, and daily tidal circulation induces accretion of the tidal marsh plain.

Individual Event Analyses - Additional insight into the existing channel performance is gained by analyzing sediment dynamics during individual hydrologic events. To assess project performance during individual events, flood hydrographs for the desired exceedance probabilities were developed. The instantaneous peak discharge exceedance probability for the Guadalupe River at San Jose gage was prepared (Figure 7-1) and provides part of the basis for development of individual flood hydrographs for a range of exceedance probabilities. Eight-day mean daily flow hydrographs were developed from gage data for the individual flood hydrograph simulations. The 8-day hydrographs were determined by developing a relationship between observed instantaneous peak discharges with observed average daily flows for flood events since the completion of the water supply reservoirs in the Guadalupe River (Figure 9-4). Significant flood events that occurred in 1980, 1982, 1986, and 1995 indicate that discharges remain at or above the discharge required to significantly mobilize the bed materials at the upstream end of the study area (~1,000 cfs) for approximately 8 days. Observed average daily discharges for these flood events are plotted on Figure 9-5. Normalizing these observed hydrographs by scaling each hydrograph by its peak average daily discharge, and then averaging the four normalized hydrographs provides a nondimensionalized hydrograph shape function that can be used to scale individual hydrographs of any desired exceedance probability. The resultant observed and average nondimensional

8 day flood hydrographs are shown on Figure 9-6. Maximum average daily discharge for any desired exceedance probability is determined by applying the power regression relationship between instantaneous and daily average peak discharge shown on Figure 9-6 for each exceedance probability of interest. Scaling this peak average daily discharge by the nondimensional 8 day flood hydrograph yields the 8 day flood hydrograph of the desired exceedance probability. Average daily flow values for the 2-, 5-, 10-, 25-, 50-, and 100-year recurrence interval are provided in Table 9-1. The resultant T-year flood hydrographs developed specifically for sedimentation analyses are shown on Figure 9-7. The 2 day 100-year recurrence interval flood hydrograph developed by the Corps of Engineers (1977) is shown for comparison on Figure 9-7. Reasonable agreement between the Corps' 2-day instantaneous hydrograph and the daily average discharge hydrograph developed by the previously described procedures is clearly evident.

Table 9-1. 8 day flood hydrograph average daily flowrates

Flowrate in cfs

Day	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
-2	106	306	576	1088	1752	2526
-1	193	559	1051	1984	3195	4607
0	621	1802	3390	6401	10307	14861
1	286	829	1559	2945	4741	6836
2	217	631	1186	2240	3607	5201
3	155	451	847	1600	2577	3715
4	112	324	610	1152	1855	2675
5	87	252	475	896	1443	2081

The HEC-6T model used the 8-day individual event hydrographs to compute sedimentation dynamics for the single event 2-, 5-, 10-, 25-, 50-, and 100-year recurrence interval floods for existing conditions. The accumulated sediment load that passes selected points within the study area for existing conditions are summarized in Table 9-2. These values represent discrete points of a probability density function for accumulated sediment load versus flood frequency. The average annual sediment load passing these selected control points can thus be approximated by numerically integrating the accumulated sediment load probability density function. The resultant average annual sediment load is also given in Table 9-2.

As was shown for the long-term simulations, Table 9-2 indicates that the fine sediment load (silt and clay) passes through the study area as wash load during the single event flood hydrographs. However, bed material from the upper watershed (sand and gravel) deposit in the study area during flood events. The magnitude of sediment delivery, defined as the accumulated sediment load passing a point over the accumulated time period since the beginning of a hydrologic event, provides a useful measure of downstream changes in sediment erosion and deposition. The sediment delivery for single hydrologic events are graphically shown in Figure 9-8 through 9-15. The figures show the accumulated sediment load at the peak of the flood hydrograph (3 days) and at the end of the event (8 days). The effects of the resultant single event sediment deposition and erosion for the 25-, 50- and 100-year hydrographs on changes in thalweg elevation and instantaneous peak 100-year flood water surface elevation is provided in Table 9-3. Water surface elevations at a specific section are a function of the channel configuration at the specific cross section as well as any backwater or drawdown affects from downstream cross sections, thus there is not always a direct correspondence between thalweg elevation change and computed water surface elevation change. Reach averaged sediment budget computations, such as those used in HEC-6T, account for this nonlinear feedback between channel geometry and water surface profiles. Figure 9-16 provides the longitudinal variation of computed instantaneous peak 100-year flood water surface elevation at selected points in the study area after the passage of the 25-, 50-, and 100-year floods. Forecast changes in the 100-year (17,000 cfs) water surface elevation following the passage of the 25-, 50- and 100-year floods (and their associated net sediment scour or accumulation) indicate that a 1 to 1.5 feet (0.3 to 0.45 m) rise in the 100-year flood water surface elevation could occur in the vicinity of Trimble to Hwy 101 due to sedimentation processes.

Table 9.2 Guad River HEC6t Run (Existing Conditions) - shows tons of sediment passing specific sections for vaious flood events

AFTER 3 DAYS (peak flow)

Bridge	Location	2 year flood		5 year flood		10 year flood		25 year flood		50 year flood		100 year flood		Average Annual	
		sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)
SPRR	22970	6	7380	17	24077	79	47988	630	92773	1766	144780	2909	167121	106.845	19257.9
Hyw 237	26343	7	7380	20	24077	143	47988	950	92773	2456	144780	3851	167121	150.145	19257.9
Tasman	30255	6	7380	32	24077	257	47988	1448	92773	3467	144780	5191	167121	216.55	19257.9
Montegue	38736	42	7380	262	24077	1136	47988	5247	92773	11348	144780	19474	167121	828.09	19257.9
Trimble	44904	632	7362	4054	24080	9441	47985	19703	92770	32912	144775	47633	167113	3751.98	19252.5
Hyw 101	47578	1339	7377	5317	24080	10524	47981	23014	92763	38997	144765	58243	167097	4686.18	19256
I-880	57896	2181	7351	5737	24053	10382	47956	17545	92713	26510	144702	34721	167003	4252.53	19236.5

AFTER 8 DAYS (end of hydrograph)

Bridge	Location	2 year flood		5 year flood		10 year flood		25 year flood		50 year flood		100 year flood		Average Annual	
		sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)
SPRR	22970	9	13270	38	45179	113	90136	684	176394	2074	283247	3694	369517	133.22	37074.9
Hyw 237	26343	16	13270	56	45179	180	90136	1040	176394	2973	283247	5077	369517	192.75	37074.9
Tasman	30255	39	13270	115	45178	300	90136	1615	176394	4344	283247	7109	369517	295.095	37074.7
Montegue	38736	117	13268	655	45178	1609	90136	6314	176394	15077	283247	26114	369517	1165.25	37074.1
Trimble	44904	1056	13268	5927	45190	13222	90132	28992	176392	50672	283242	76750	369510	5630.97	37075.9
Hyw 101	47578	2162	13268	8526	45188	15815	90128	33484	176384	57093	283232	86527	369490	7112.66	37074.4
I-880	57896	3473	13228	9794	45154	17954	90102	34233	176329	52012	283164	73634	369392	7691.03	37048.9

Table 9-3. Guadalupe River HEC6t Run (existing conditions) - thalweg & wsp for Q=17,000cfs before and after 8 day event

Elevation Difference in Feet

		before event		after 25 year flood				after 50 year flood				after 100 year flood			
Bridge	Location	before thalweg	before wsp	after thalweg	diff ta-tb	after wsp	diff ha-hb	after thalweg	diff ta-tb	after wsp	diff ha-hb	after thalweg	diff ta-tb	after wsp	diff ha-hb
SPRR	22970	-7.48	16.46	-7.48	0	16.46	0	-7.48	0	16.46	0	-7.48	0	16.47	0.01
Hwy 237	26343	0.2	19.76	0.2	0	19.76	0	0.2	0	19.76	0	0.2	0	19.76	0
Tasman	30255	-2.03	23.23	-2.03	0	23.24	0.01	-2.03	0	23.25	0.02	-2.03	0	23.26	0.03
Montegue	38736	2.62	26.48	2.76	0.14	26.75	0.27	2.76	0.14	26.77	0.29	2.76	0.14	26.81	0.33
Trimble	44904	14.34	30.01	13.69	-0.65	30.18	0.17	13.35	-0.99	30.29	0.28	13.11	-1.23	30.43	0.42
Hwy 101	47578	19.61	32.25	18.1	-1.51	33.4	1.15	17.29	-2.32	33.54	1.29	16.43	-3.18	33.79	1.54
I-880	57896	37.47	56.81	37.79	0.32	57.13	0.32	37.46	-0.01	56.19	-0.62	37.47	0	55.39	-1.42

Elevation Difference in Meters

		before event		after 25 year flood				after 50 year flood				after 100 year flood			
Bridge	Location	before thalweg	before wsp	after thalweg	diff ta-tb	after wsp	diff ha-hb	after thalweg	diff ta-tb	after wsp	diff ha-hb	after thalweg	diff ta-tb	after wsp	diff ha-hb
SPRR	22970	-2.28	5.02	-2.28	0.00	5.02	0.00	-2.28	0.00	5.02	0.00	-2.28	0.00	5.02	0.00
Hwy 237	26343	0.06	6.02	0.06	0.00	6.02	0.00	0.06	0.00	6.02	0.00	0.06	0.00	6.02	0.00
Tasman	30255	-0.62	7.08	-0.62	0.00	7.08	0.00	-0.62	0.00	7.09	0.01	-0.62	0.00	7.09	0.01
Montegue	38736	0.80	8.07	0.84	0.04	8.15	0.08	0.84	0.04	8.16	0.09	0.84	0.04	8.17	0.10
Trimble	44904	4.37	9.15	4.17	-0.20	9.20	0.05	4.07	-0.30	9.23	0.09	4.00	-0.37	9.28	0.13
Hwy 101	47578	5.98	9.83	5.52	-0.46	10.18	0.35	5.27	-0.71	10.22	0.39	5.01	-0.97	10.30	0.47
I-880	57896	11.42	17.32	11.52	0.10	17.41	0.10	11.42	-0.00	17.13	-0.19	11.42	0.00	16.88	-0.43

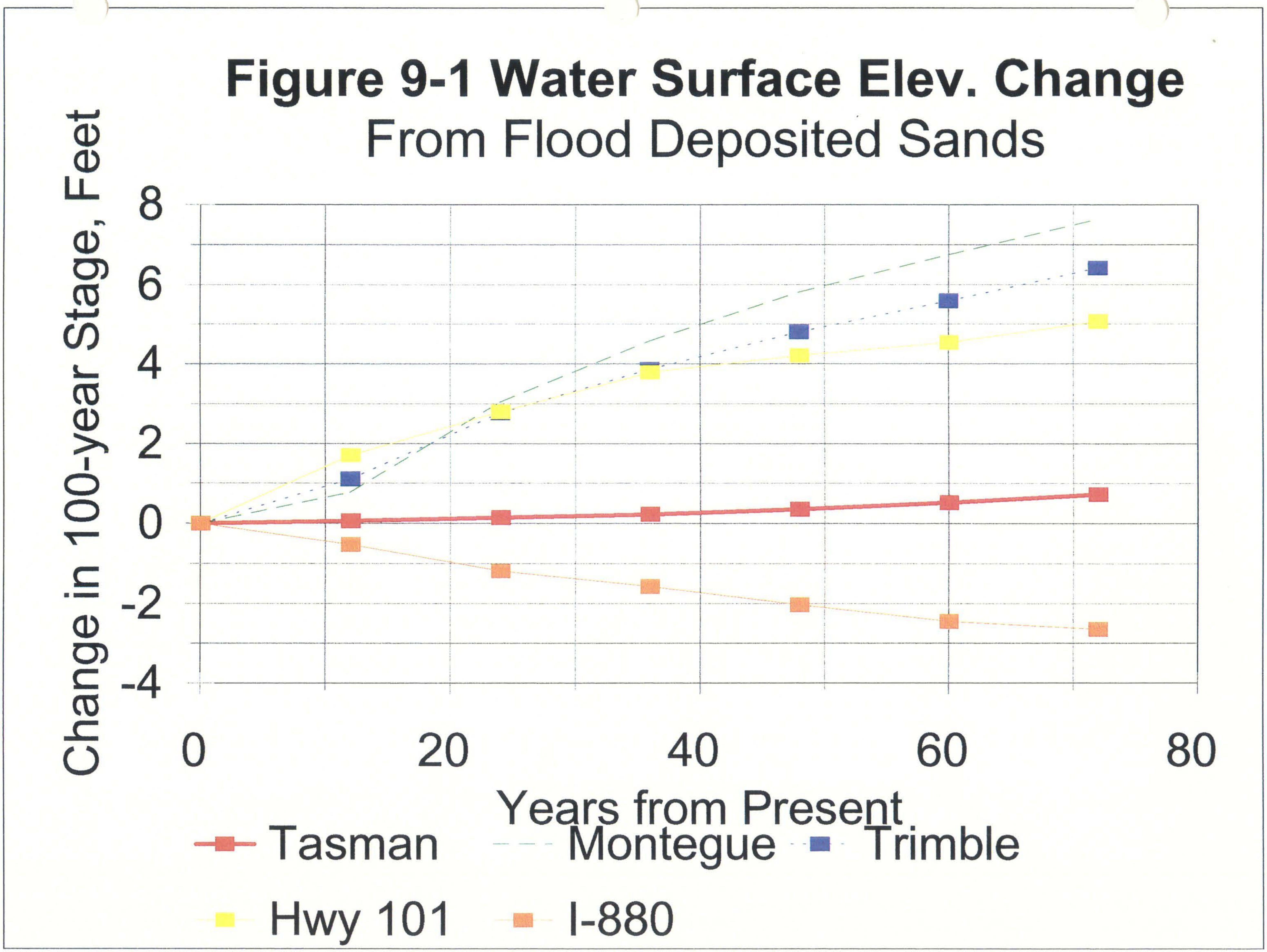


Figure 9-1 Water Surface Elev. Change

From Flood Deposited Sands

Figure 9-2 Water Surface Elev. Change
From Flood Deposited Sands

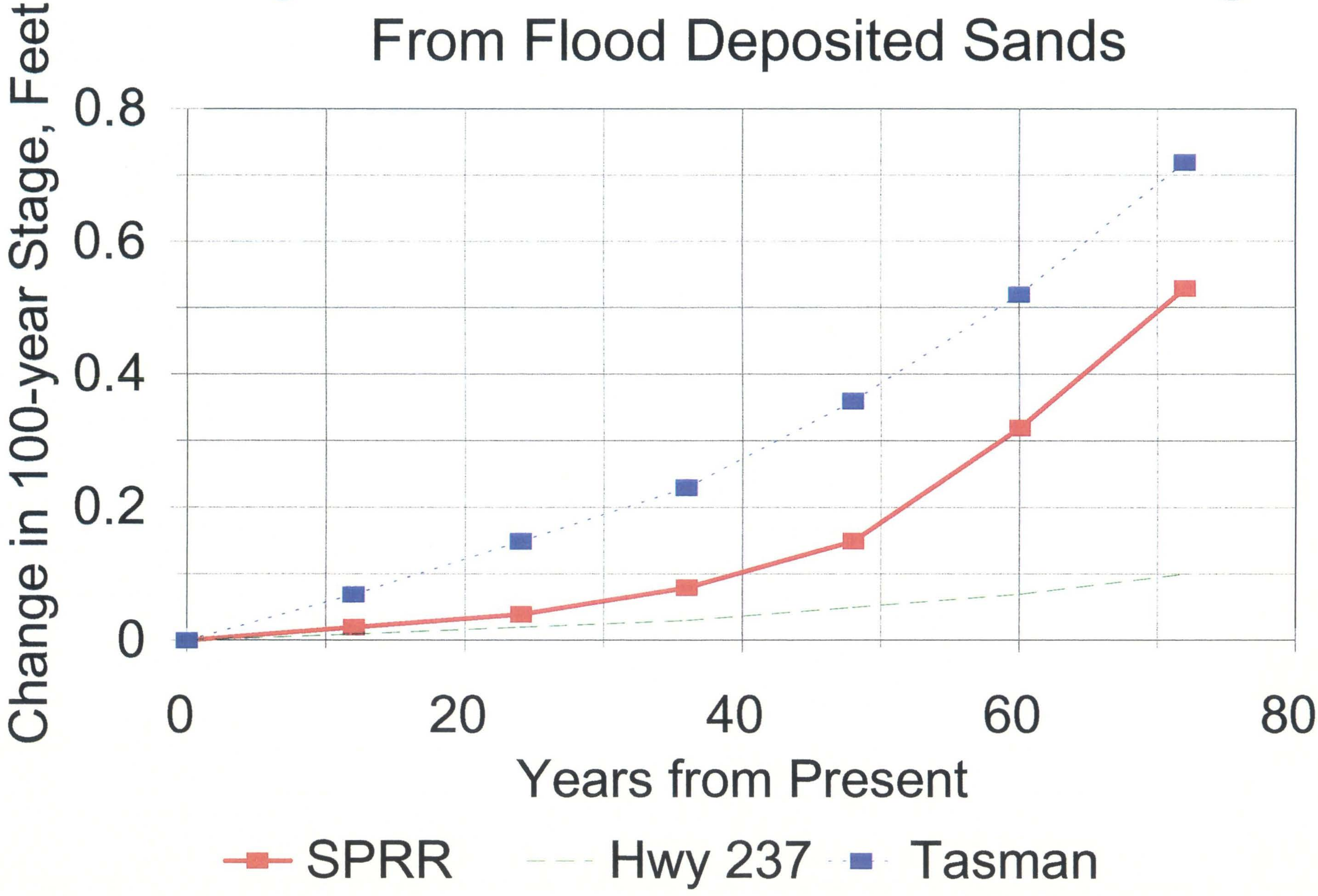
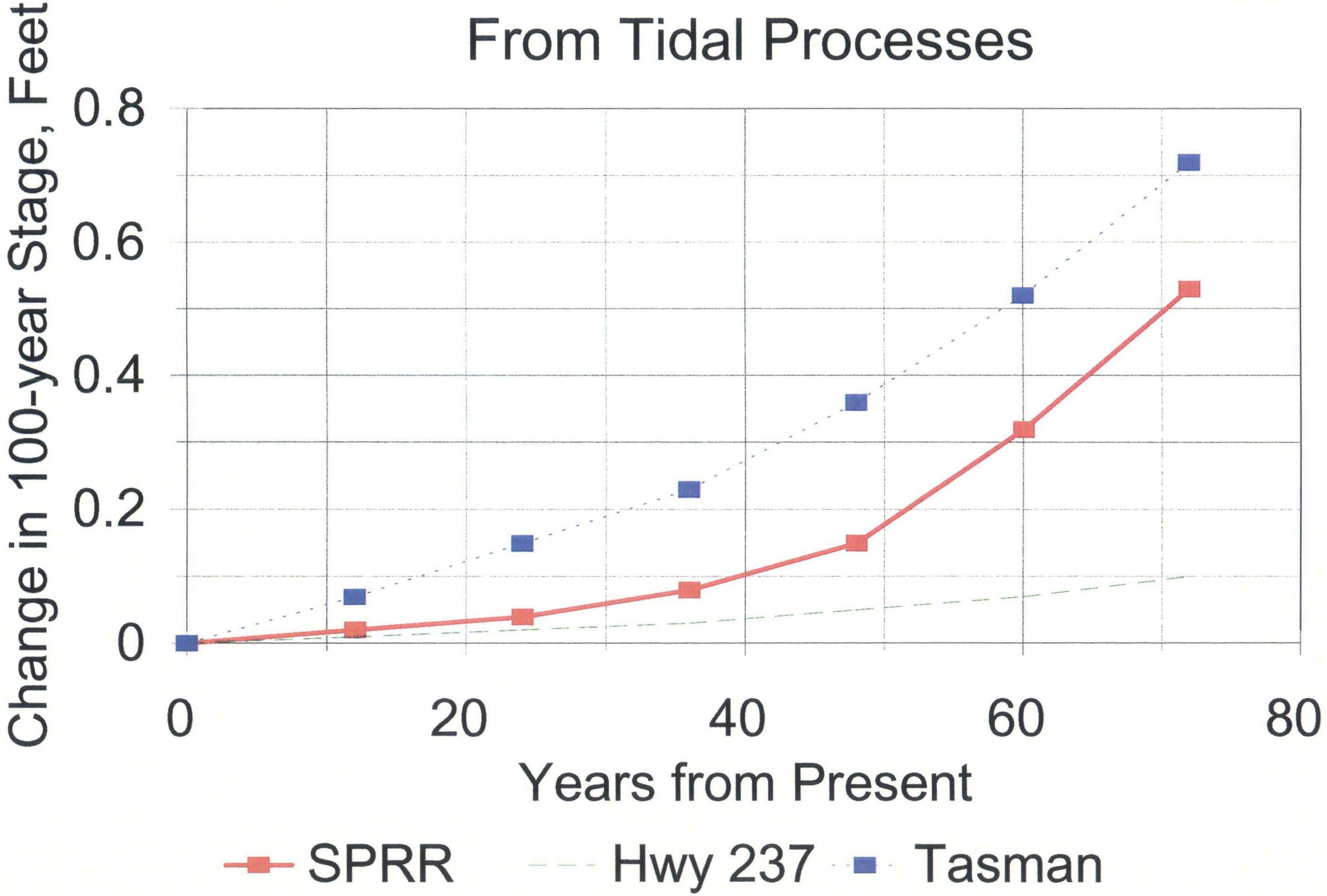


Figure 9-3 Water Surface Elev. Change
From Tidal Processes



Guadalupe River Hydrology

Peak flow vs. avg daily flow, 1953-95

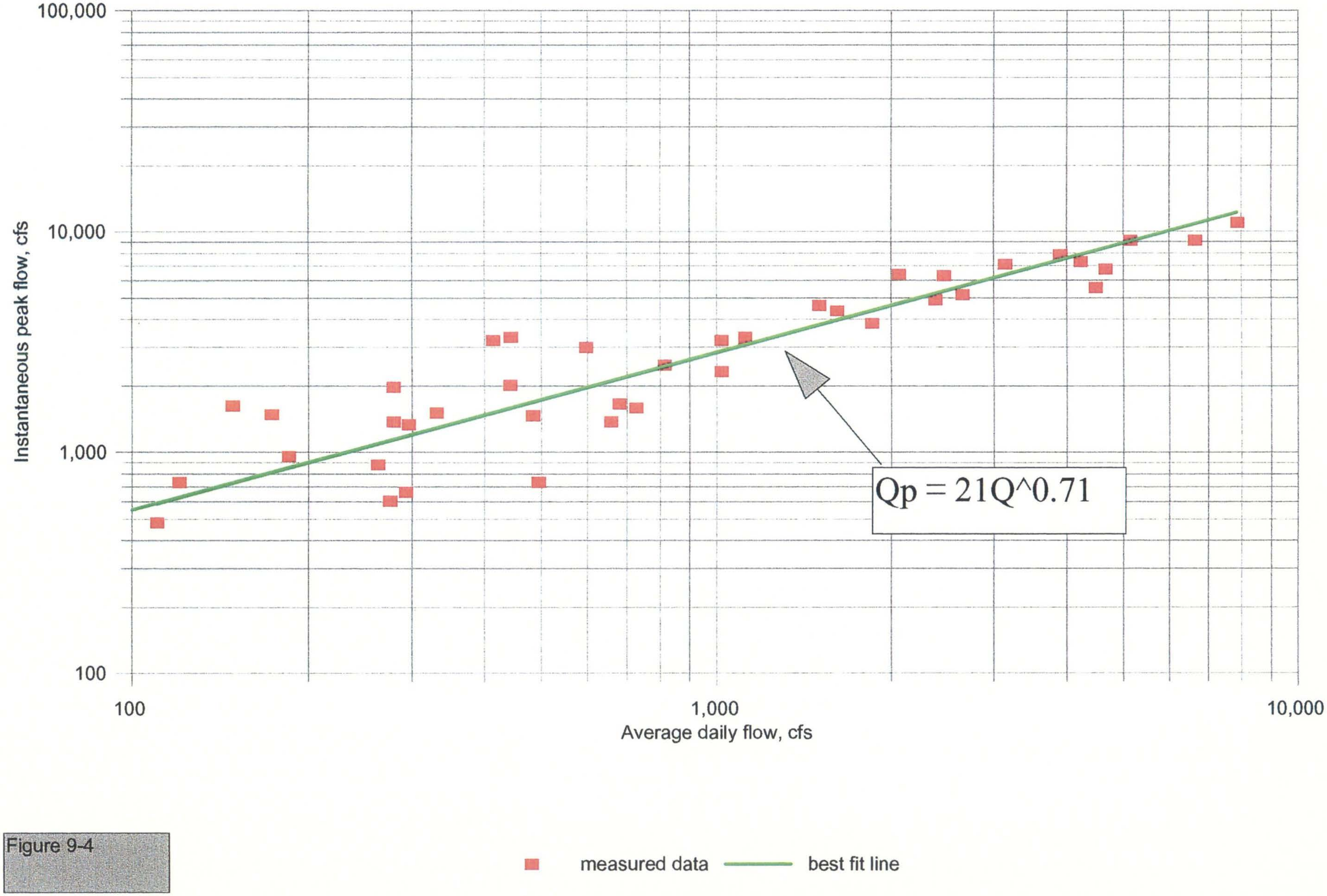


Figure 9-4

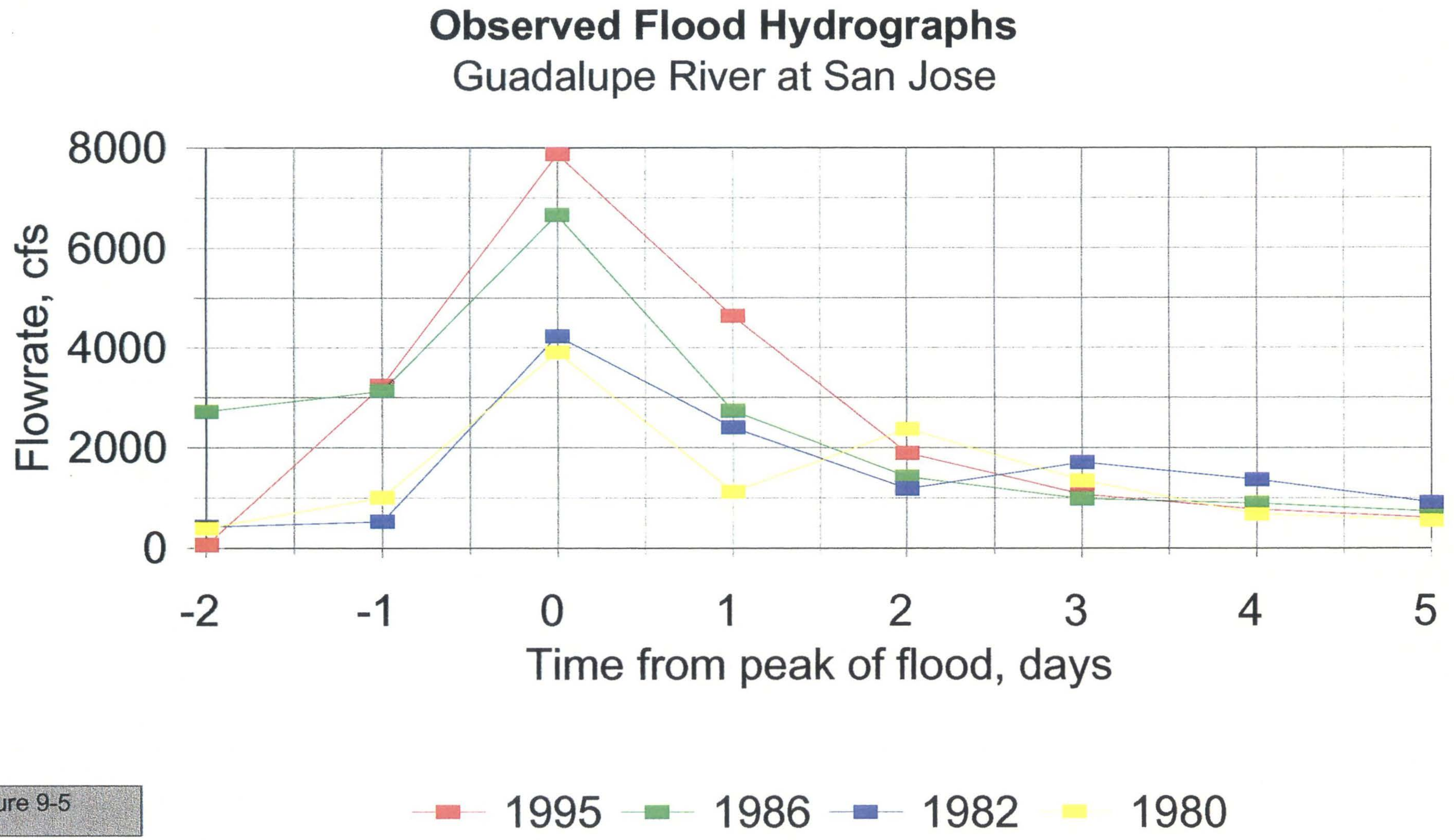

Observed Flood Hydrographs
Guadalupe River at San Jose
8000
6000
4000
2000
0
Flowrate, cfs
-2
-1
0
1
2
3
4
5
Time from peak of flood, days
1995
1986
1982
1980

Figure 9-5

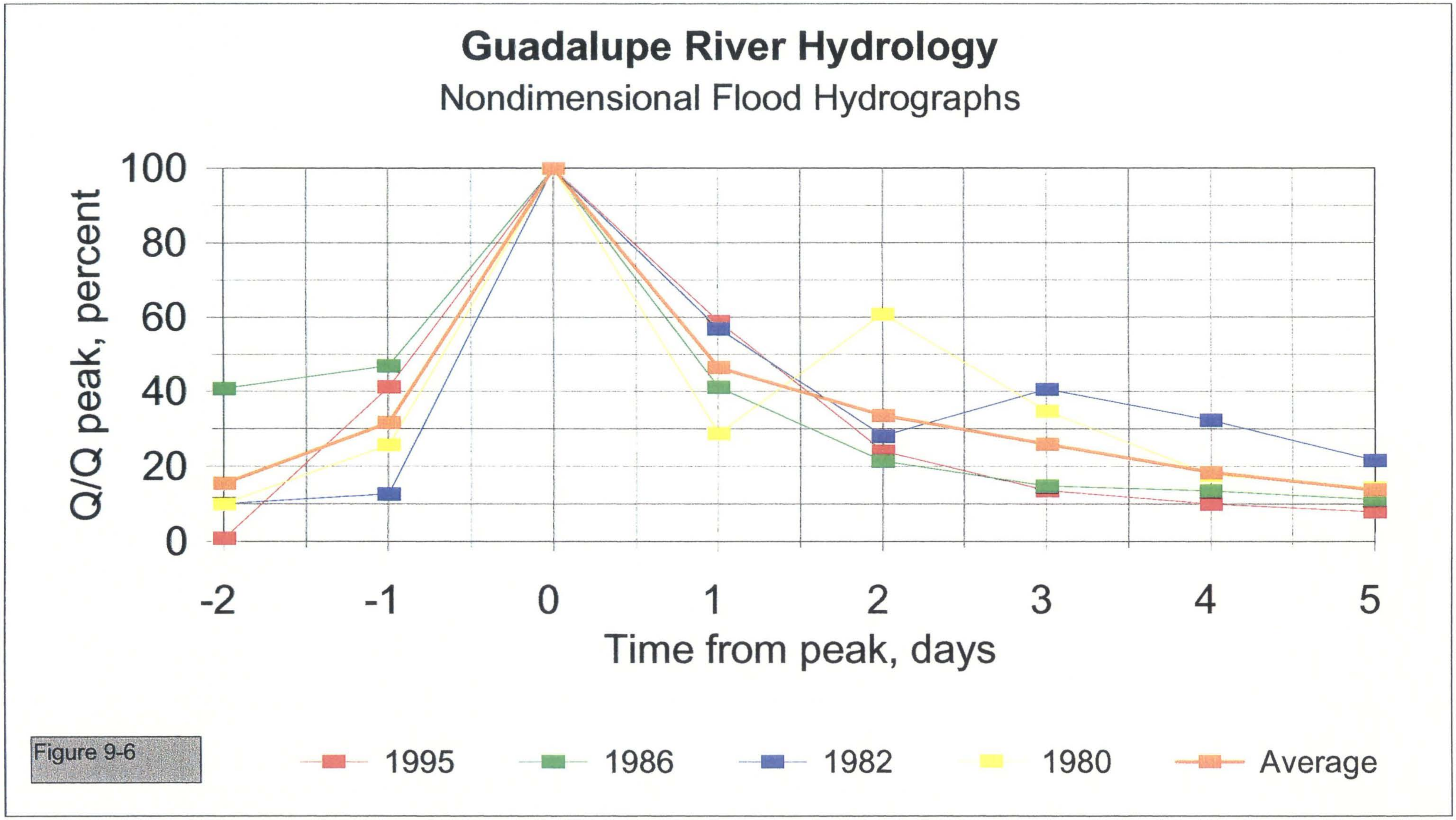

Guadalupe River Hydrology
Nondimensional Flood Hydrographs
Q/Q peak, percent
100
80
60
40
20
0
-2
-1
0
1
2
3
4
5
Time from peak, days
1995
1986
1982
1980
Average

Figure 9-6

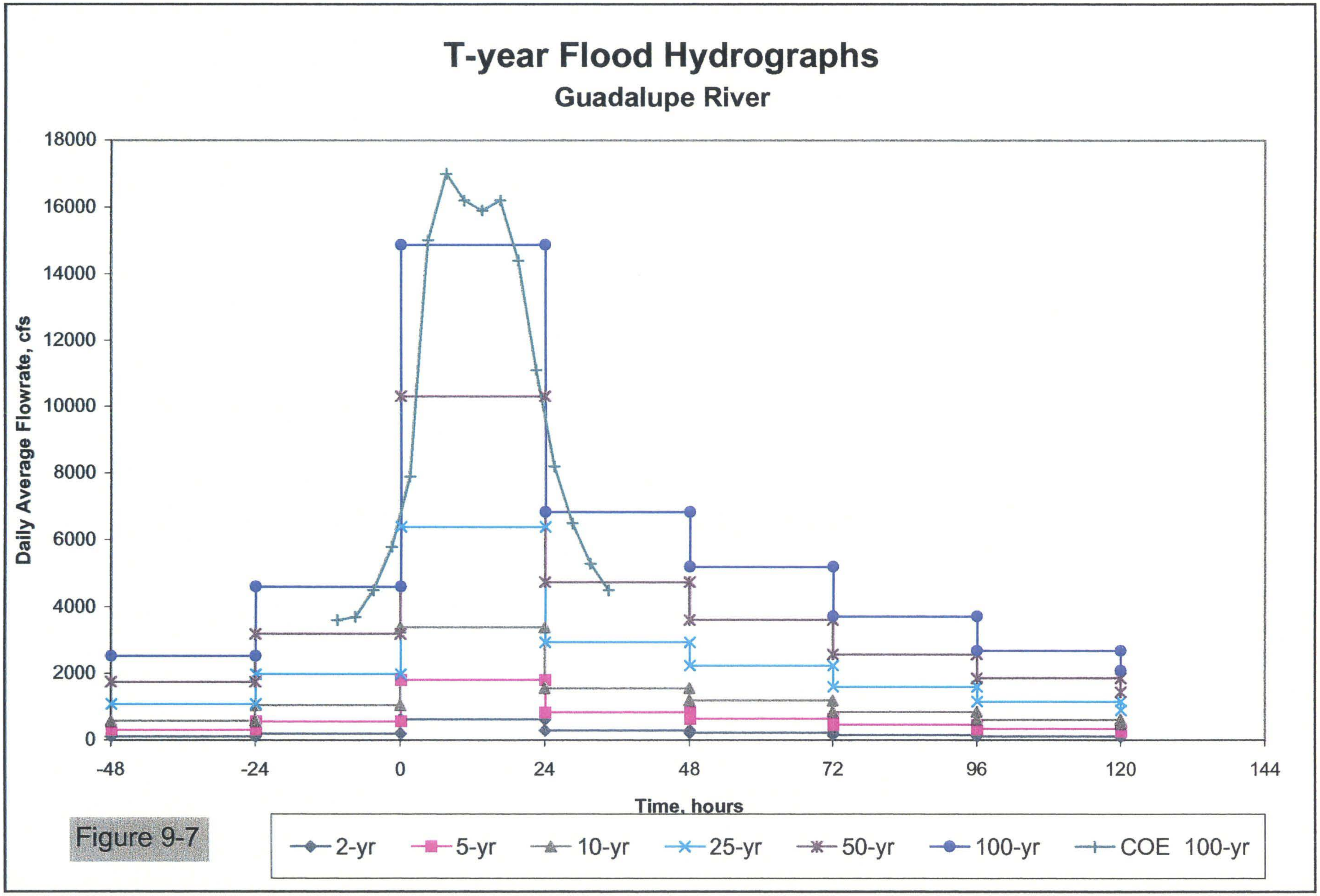

T-year Flood Hydrographs
Guadalupe River
Daily Average Flowrate, cfs
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
-48
-24
0
24
48
72
96
120
144
Time, hours
2-yr
5-yr
10-yr
25-yr
50-yr
100-yr
COE 100-yr

Figure 9-7

Sediment Delivery

2-year flood (existing cond)

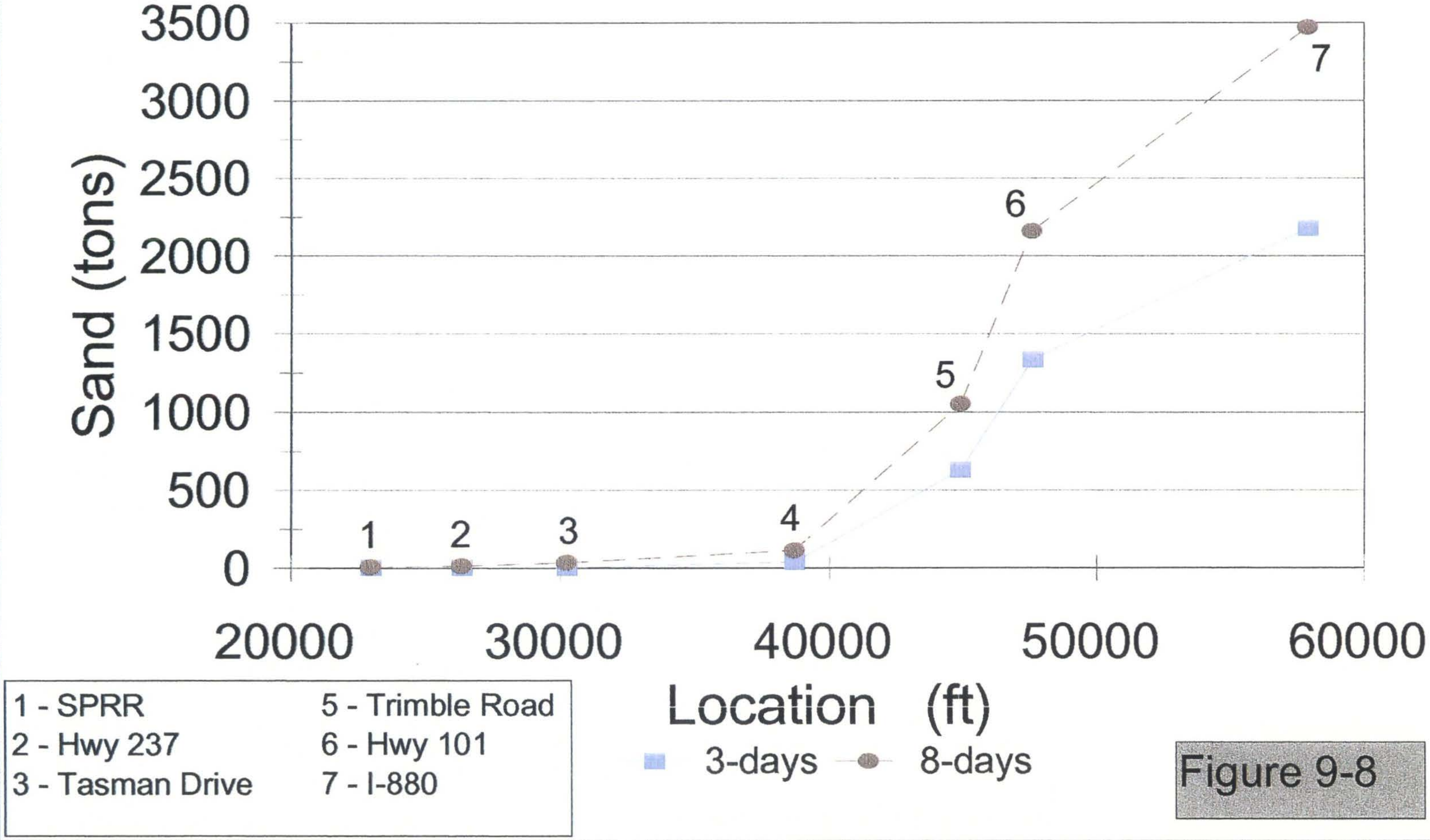
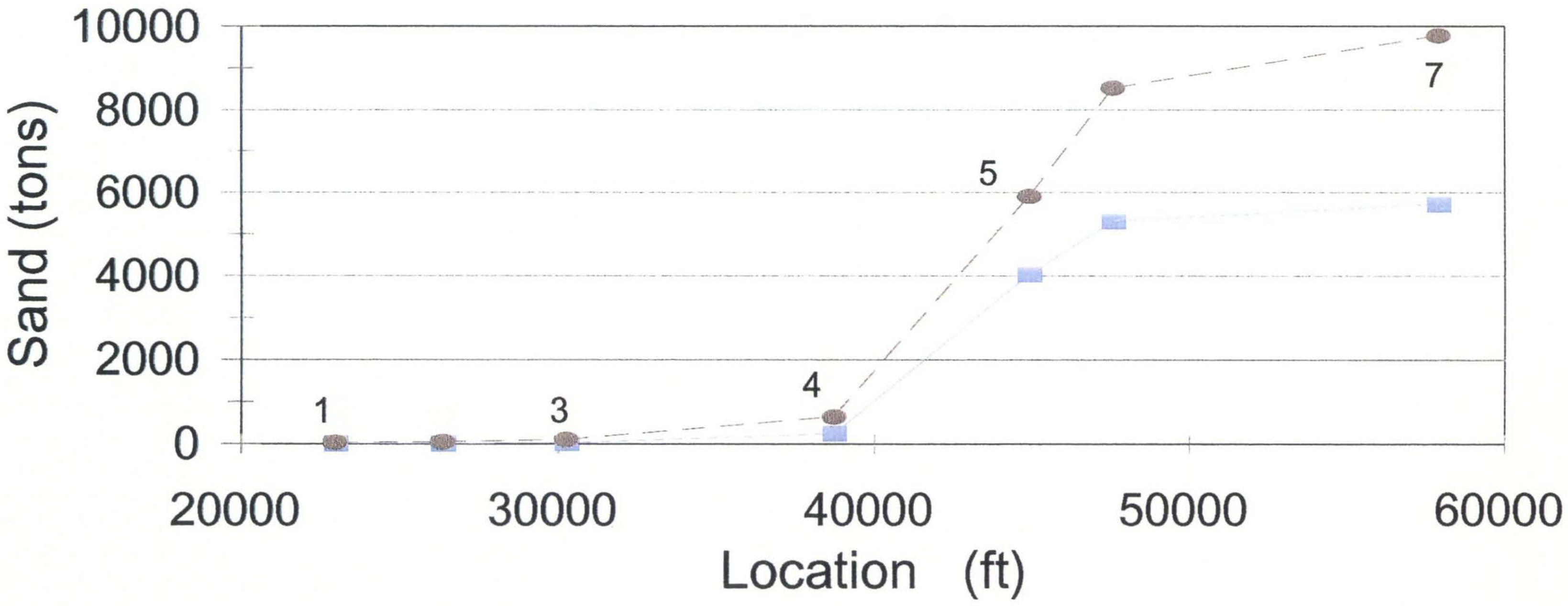


Figure 9-8

Sediment Delivery

5-year flood (existing cond)



1 - SPRR
2 - Hwy 237
3 - Tasman Drive
5 - Trimble Road
6 - Hwy 101

■ 3-days ● 8-days

Figure 9-9

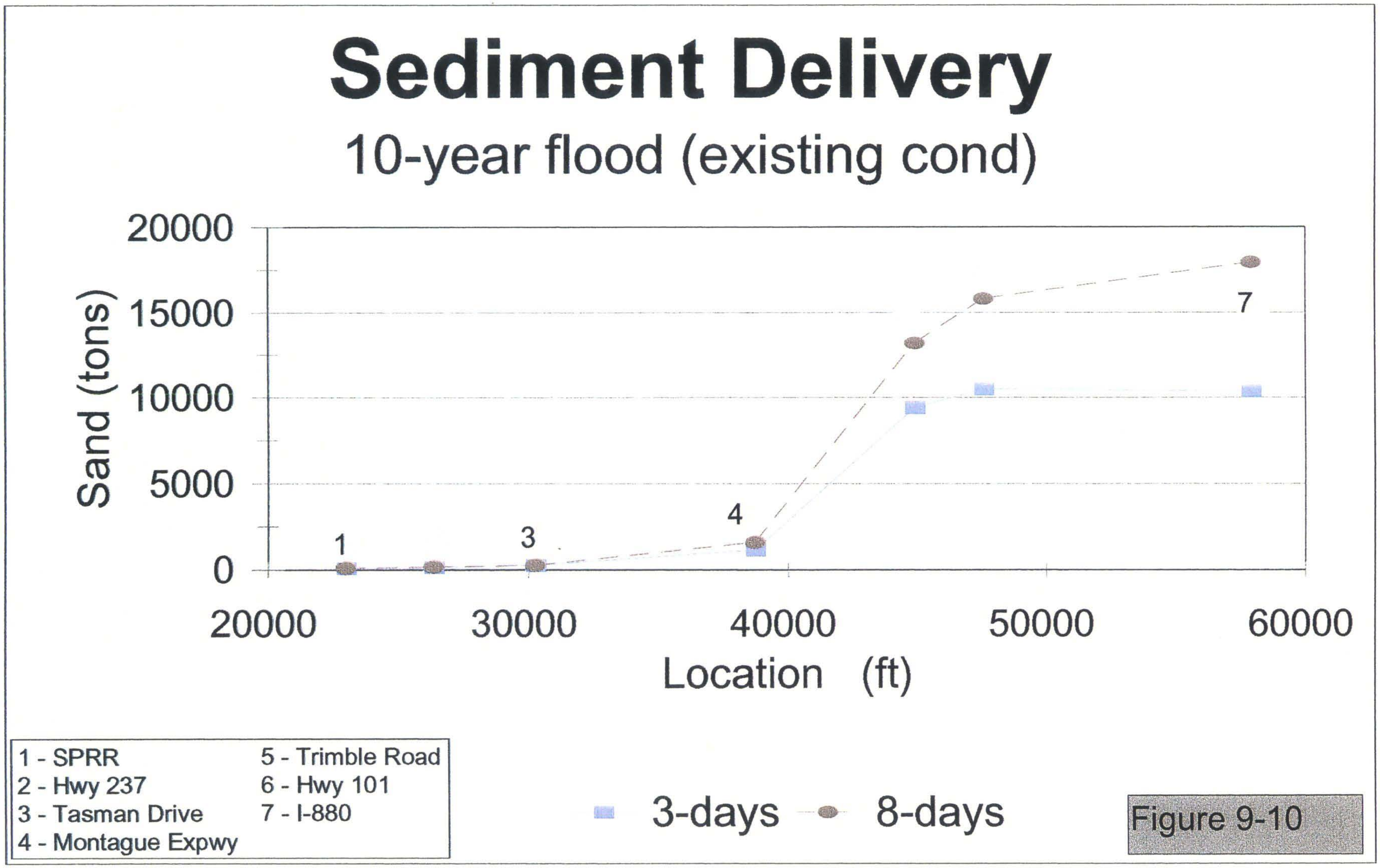

Sediment Delivery
10-year flood (existing cond)
Sand (tons)
20000
15000
10000
5000
0
20000
30000
40000
50000
60000
Location (ft)
1
3
4
7
1 - SPRR
2 - Hwy 237
3 - Tasman Drive
4 - Montague Expwy
5 - Trimble Road
6 - Hwy 101
7 - I-880
3-days
8-days

Figure 9-10

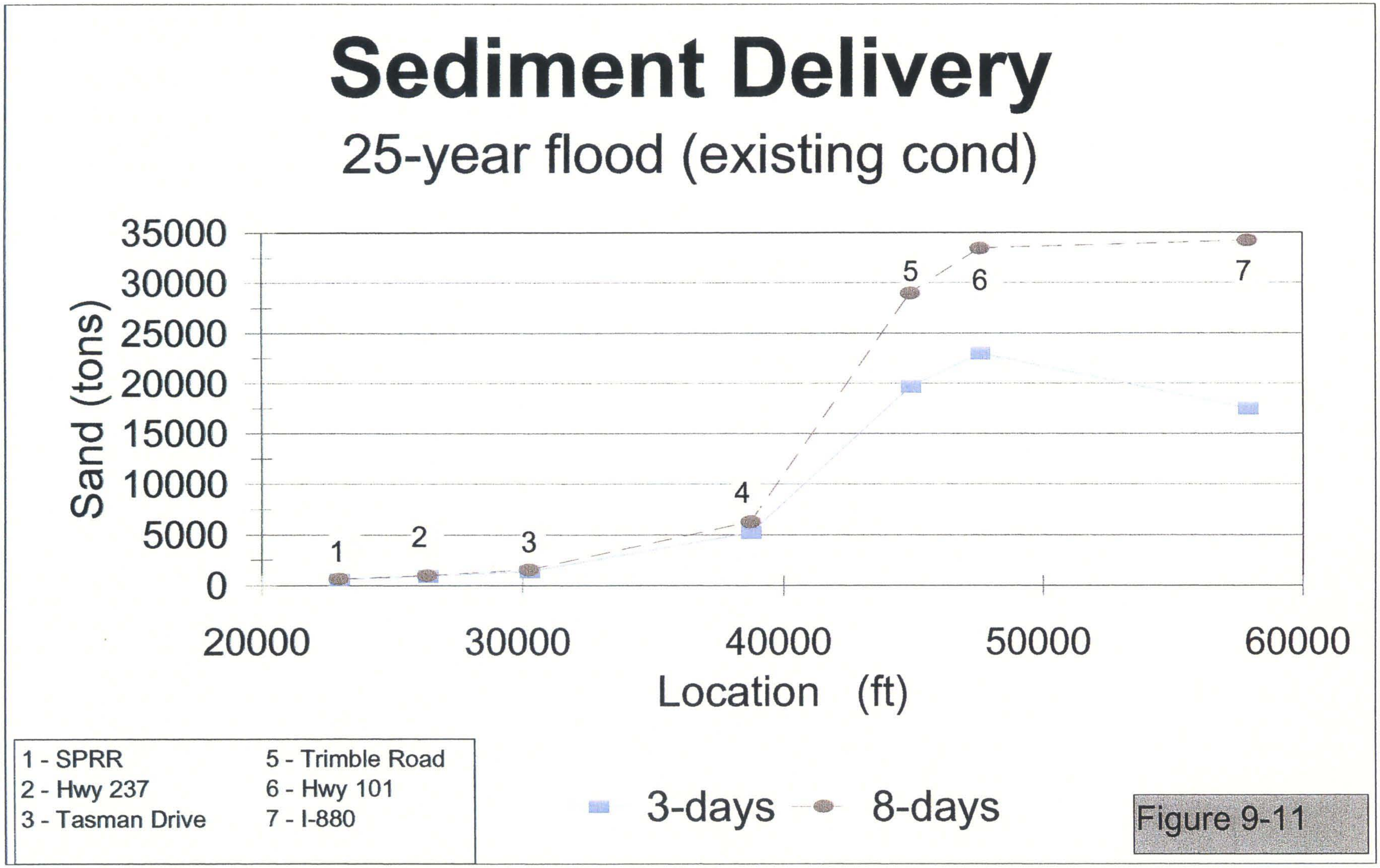

Sediment Delivery
25-year flood (existing cond)
Sand (tons)
35000
30000
25000
20000
15000
10000
5000
0
20000
30000
40000
50000
60000
Location (ft)
1
2
3
4
5
6
7
1 - SPRR
2 - Hwy 237
3 - Tasman Drive
5 - Trimble Road
6 - Hwy 101
7 - I-880
3-days
8-days

Figure 9-11

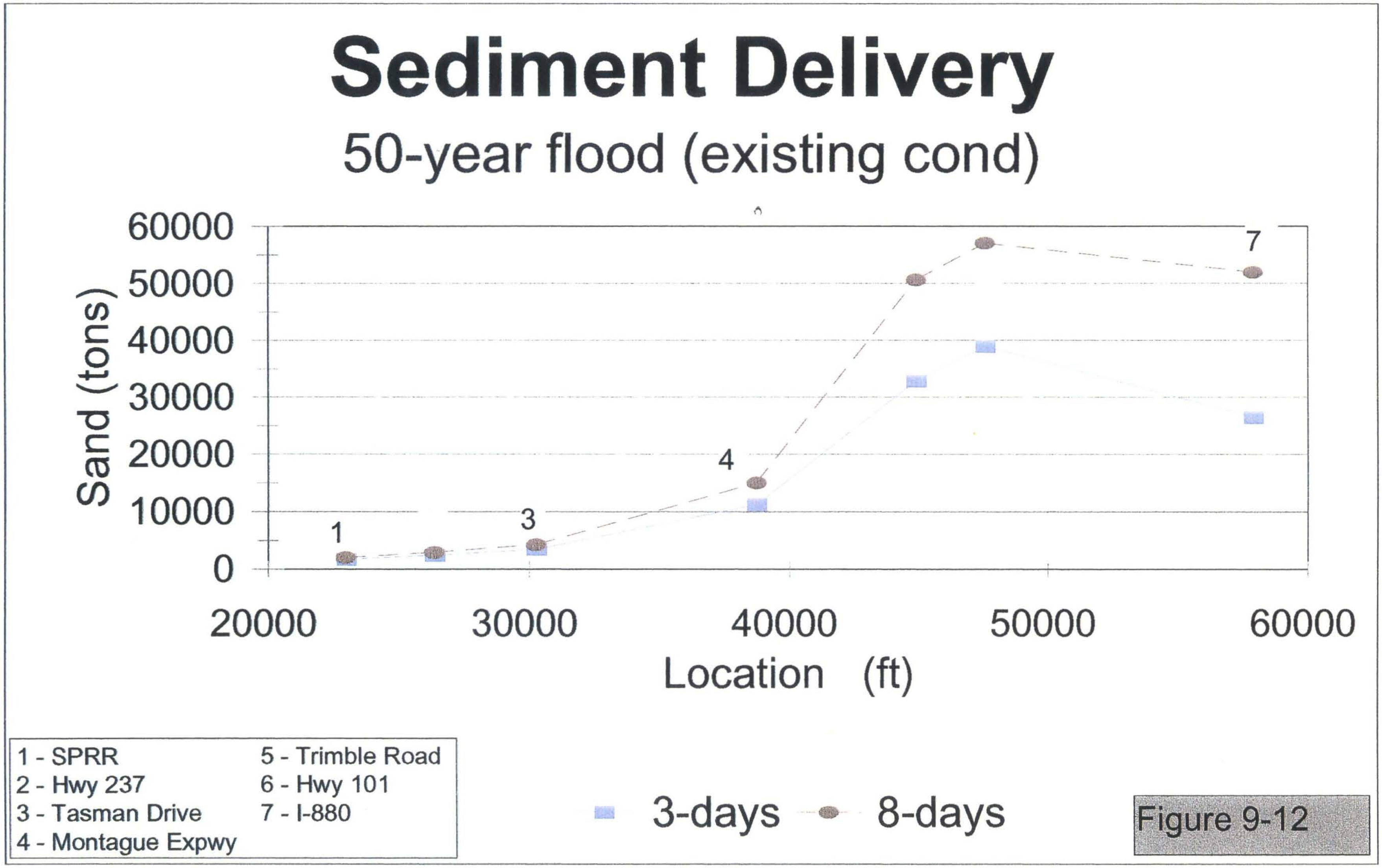

Sediment Delivery
50-year flood (existing cond)
Sand (tons)
60000
50000
40000
30000
20000
10000
0
20000
30000
40000
50000
60000
Location (ft)
1
3
4
7
1 - SPRR
2 - Hwy 237
3 - Tasman Drive
4 - Montague Expwy
5 - Trimble Road
6 - Hwy 101
7 - I-880
3-days
8-days

Figure 9-12

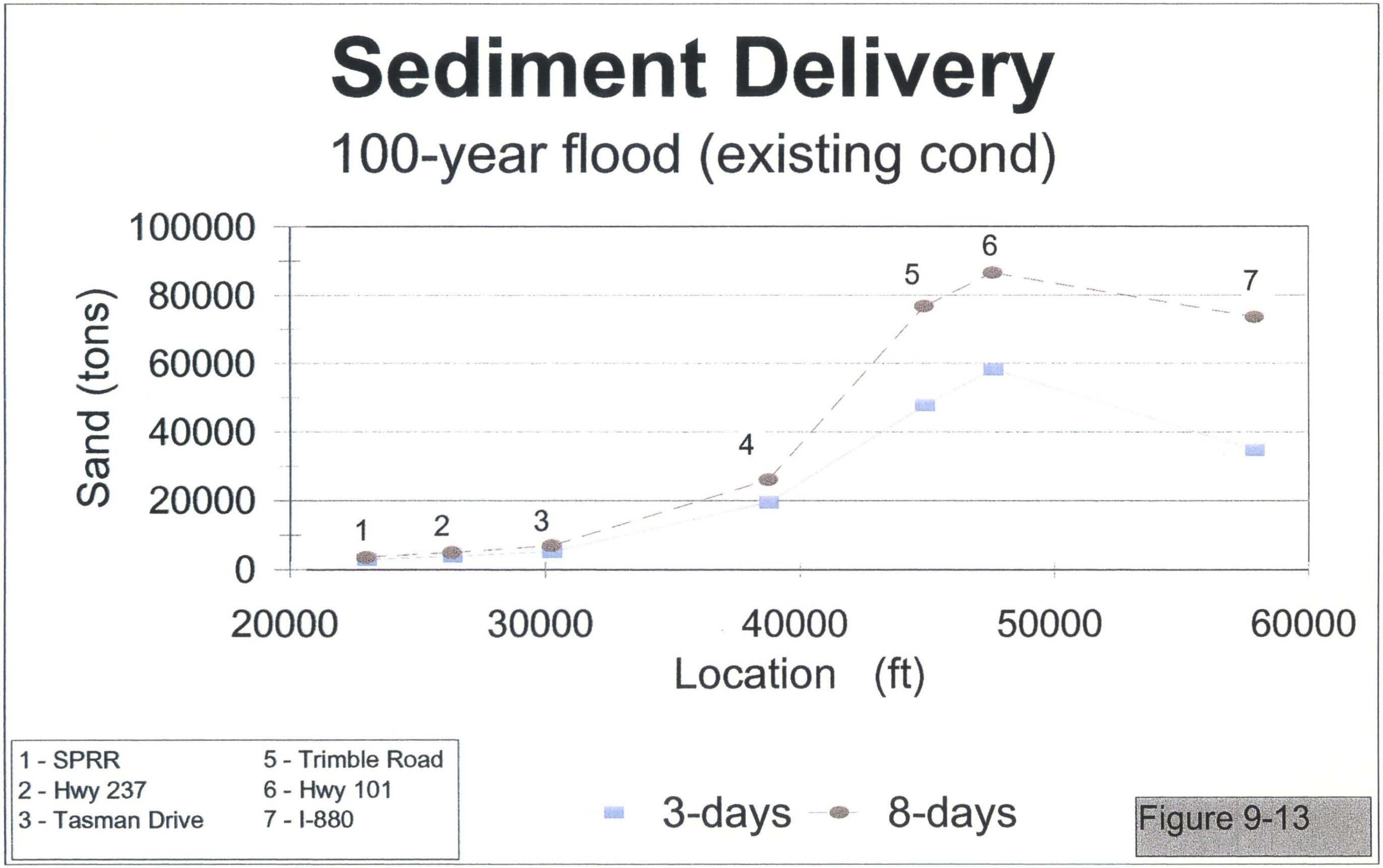
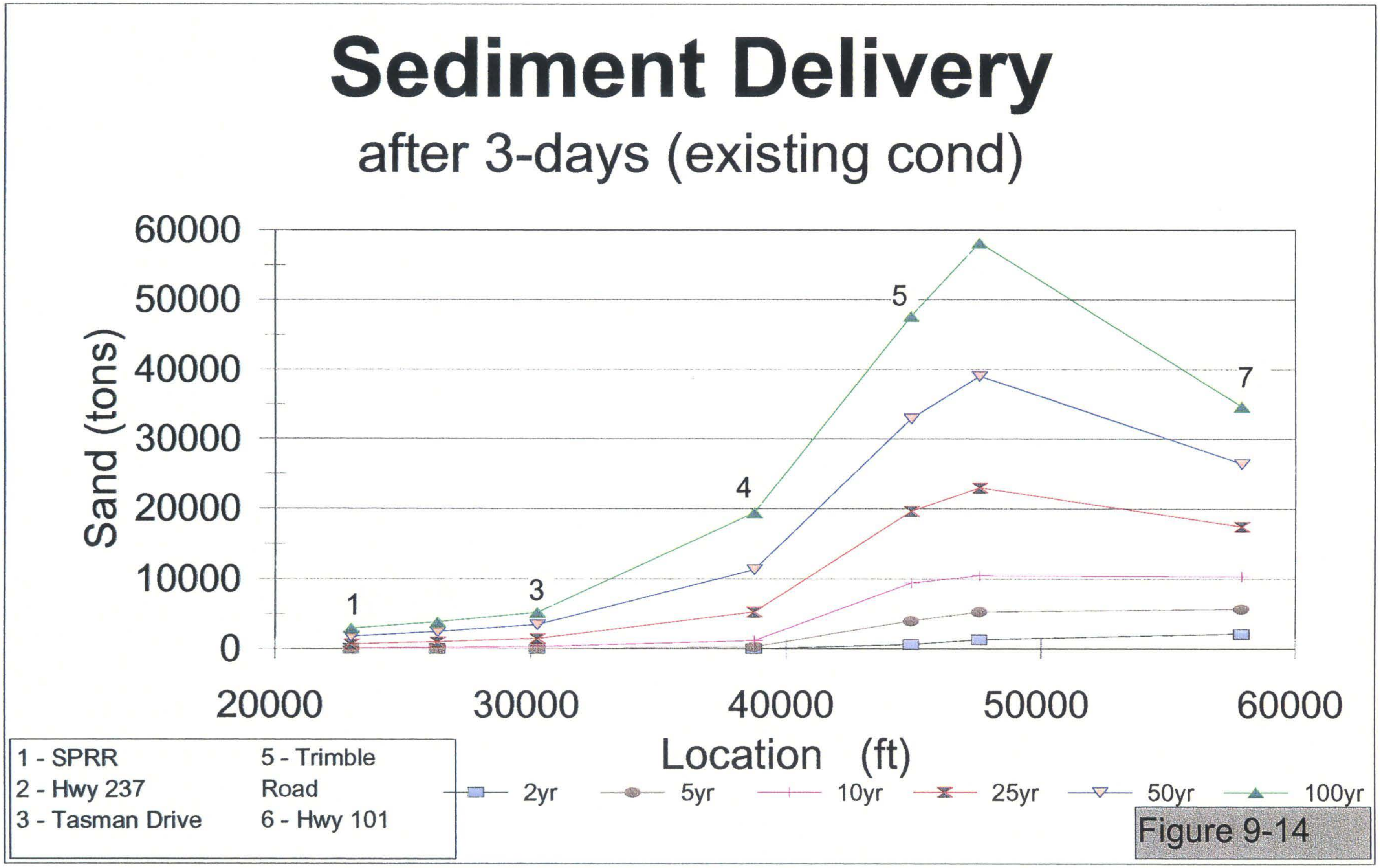

Sediment Delivery
100-year flood (existing cond)
Sand (tons)
100000
80000
60000
40000
20000
0
20000
30000
40000
50000
60000
Location (ft)
1
2
3
4
5
6
7
1 - SPRR
2 - Hwy 237
3 - Tasman Drive
5 - Trimble Road
6 - Hwy 101
7 - I-880
3-days
8-days

Figure 9-13



Sediment Delivery

after 3-days (existing cond)

Figure 9-14

Sediment Delivery

after 8-days (existing cond)

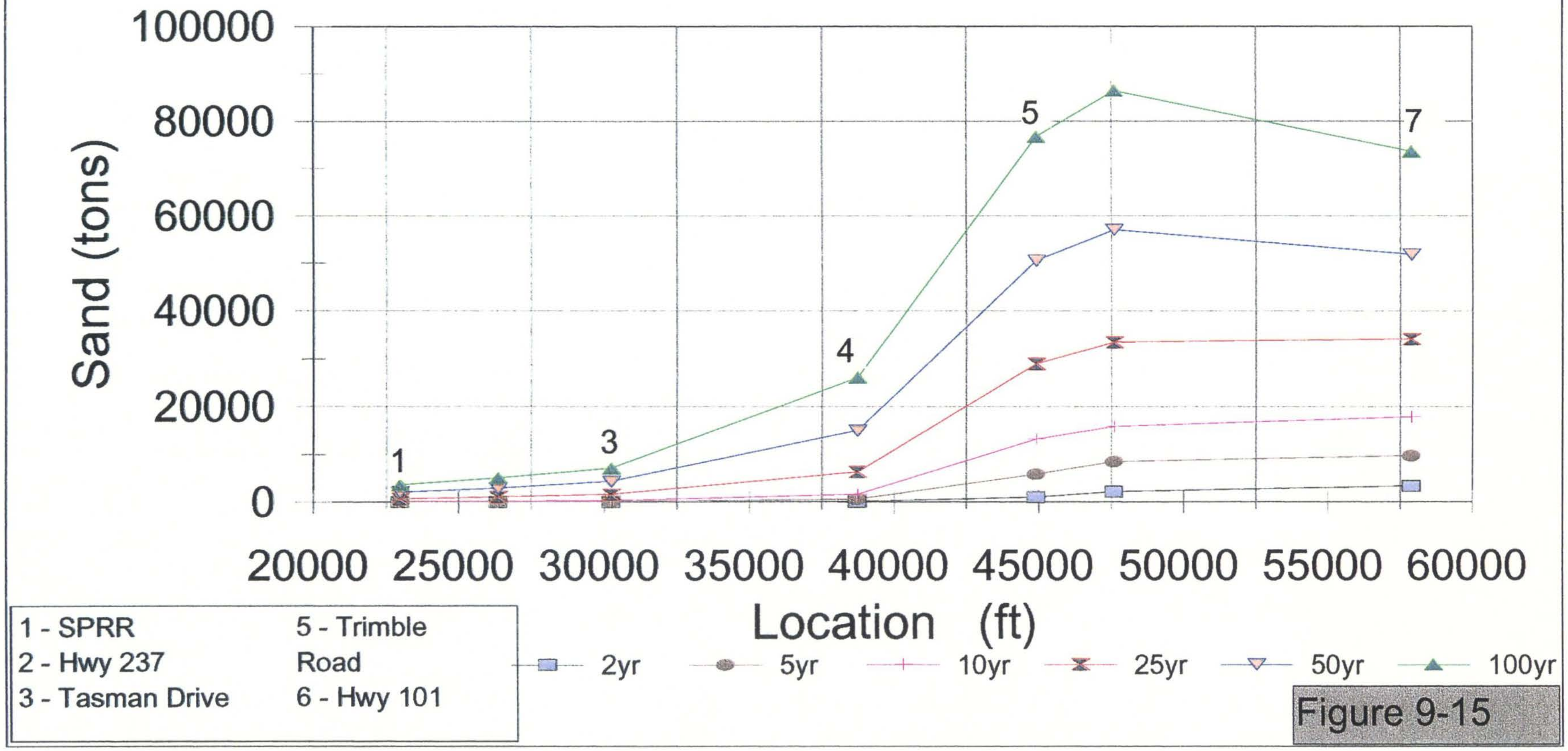
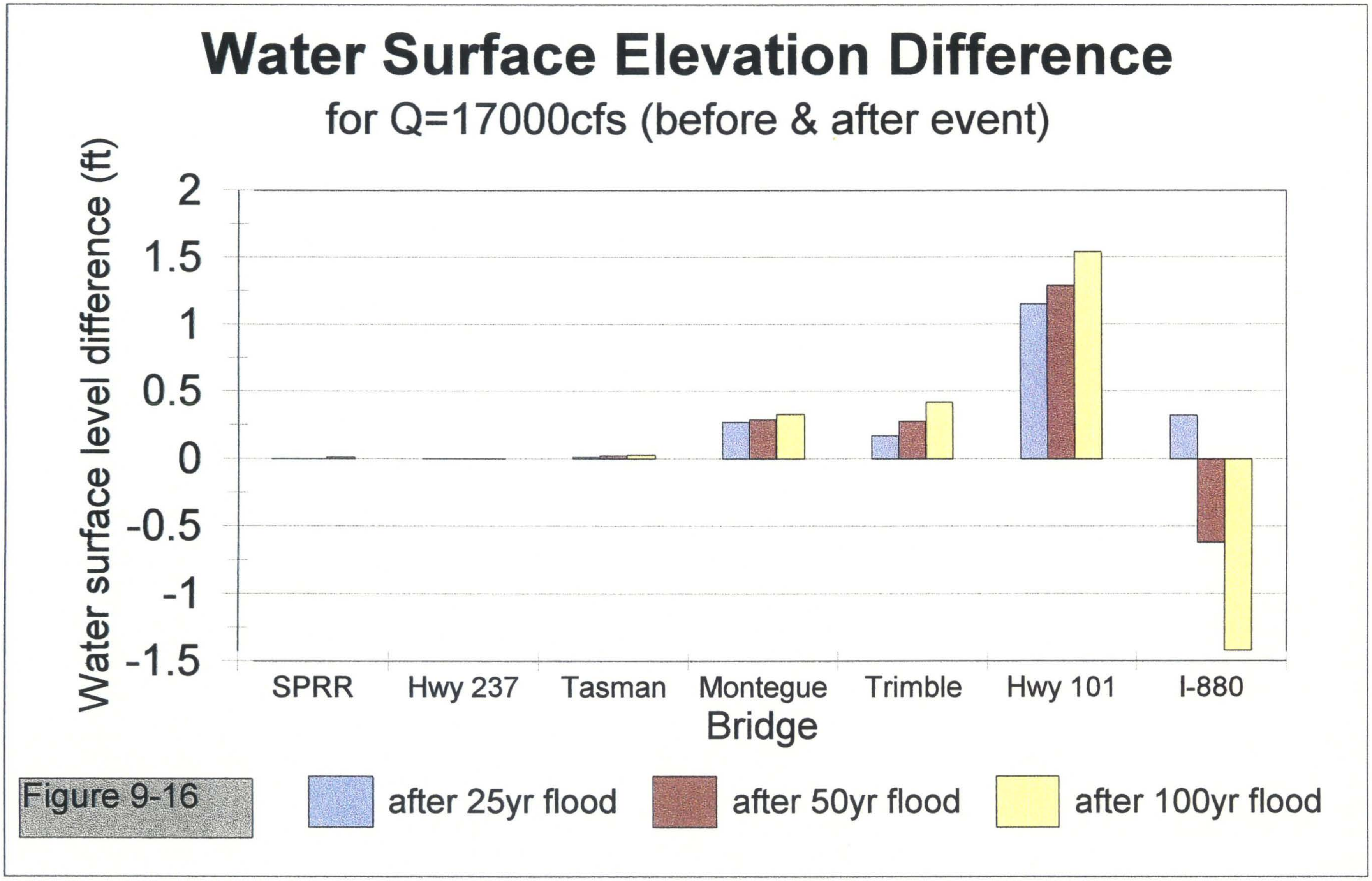


Figure 9-15



Water Surface Elevation Difference

for Q=17000cfs (before & after event)

Figure 9-16

Chapter 10

Sedimentation Analysis of Project Alternatives

10. Sedimentation Analysis of Project Alternatives

Sediment and channel management alternatives being considered by the District include vegetation maintenance (vegetation removal as well as the needs for additional mitigation plantings), channel and/or overbank excavation, levee or flood wall raising, and modifications to the existing bridges. As an initial assessment of the affects of channel maintenance activities on sedimentation management, NHC performed preliminary analyses of project alternatives and presented the results of these initial studies to the District in October 1997. This initial assessment assumed increased roughness of the overbank portions of the channel to reflect increased vegetation growth and density. Preliminary results presented to the District indicated that increasing overbank channel roughness due to densification of vegetation affects the rate and location of sediment deposition and tends to extend zones of deposition further upstream from that which occurs under existing conditions. It was, therefore, determined that channel hydraulics and sediment dynamics are sensitive to the location, height and density of in-channel vegetation.

Four types of project alternatives for six subreaches of the river were prescribed by the District. A description of the types of management alternatives is provided in Table 10-1. The locations of channel maintenance subreaches are indicated in Figure 10-1. Alternative 1 proposes no change over existing channel maintenance activities, and thus reflects the existing conditions described previously in Chapter 9. Alternatives 2 and 3 proposed different types of in-channel vegetation maintenance, while Alternative 4 repeats history by excavating the channel back to the 1983 condition, and continuing with the existing channel maintenance program. For Alternative 4 a channel response similar to that observed since the 1983 channel maintenance would thus be expected. All in-channel vegetation management alternatives were accompanied with similar capital improvement assumptions such as raising the levees and modifying bridge constrictions to contain the design event. Detailed analyses of site specific capital improvement projects were not evaluated.

Alternatives 2 and 3 reflect different magnitudes of vegetation maintenance, with the goal of reducing hydraulic roughness, and therefore increasing flow efficiency throughout the project reach. This was accomplished by reducing the overbank Manning's n value by an appropriate factor to account for the proposed vegetation maintenance activity. The District developed guidelines for appropriate values of Manning's n as a function of vegetation type and density. These values are shown in Table 7-2 in Chapter 7. Based on the guidance provided in this table, overbank Manning's n values were reduced to a maximum of 0.06 to 0.08 in the project reach. In addition, Alternative 3 implements limited overbank excavation in the reach between Trimble Road and Montague Avenue, to provide a potential location for managed sediment deposition and removal. Therefore, Alternatives 2 and 3 were evaluated with HEC-6T by reducing the overbank Manning's n values, according to District guidelines and in the case of Alternative 3, modifying overbank channel geometry to reflect the presence, and maintenance of sediment storage basins. The intent of the sediment management basins is to determine if "off channel" sediment storage and periodic removal is effective. The sediment basins are depicted by excavating a 4-foot deep channel template through the right overbank portion of the this sub reach. Channel excavation was not specified at the upstream and downstream ends of the subreach to depict a closed basin and the off channel hydraulics of such a configuration.

Long Term Trends - Long term simulations (i.e. > 50 years) with the HEC-6T model allow the comparison of present and future project performance with each of these project alternatives in place. The same hydrologic conditions used for long term simulations presented in Chapter 9 for existing conditions were used here to evaluate long term (up to 72 years) sediment dynamics in the study reach.

Changes in the computed 100-year (0.01 probability of exceedence) flood water surface elevation for existing conditions (Alternative 1) are presented in Chapter 9, Figures 9-1 through 9-3. Changes in the 100-year water surface elevation for Alternatives 2 and 3 are shown in Figures 10-2 through 10-5. Reach averaged sediment storage dynamics with the project Alternatives 2 and 3 in place are essentially unchanged from existing conditions, as all flood derived sediments continue to deposit in the same area of the project reach, however, the deposits are shifted slightly downstream. The locations of sediment deposition are changed slightly from existing conditions as a greater percentage of the sediment is deposited at locations downstream from that determined for existing conditions. This downstream shift in location of sediment storage results from the decreased roughness and frictional losses associated with the vegetation management activities proposed by the project condition alternatives. Shifting the aggradation downstream aggravates existing channel capacity limitations in the lower reaches.

Results show that the prescribed "off channel" sediment basins are only marginally effective at preferentially capturing fine sand sized sediments, however, the abilities of the 1-dimensional HEC-6T model for simulating the relatively complex 3-dimensional processes of sediment diversions at flow bifurcations and sediment basins is limited (Hall and Gessler, 1997).

Project Alternatives 2 and 3 have negligible influence during flood events on the depositional regime of suspended silts and clays. Fine sediments are transported through the project reach as wash load and any deposition of suspended silt and clay occurs in the lowest reaches of the project area which are subject to tidal influences. Transport of additional sediment materials to downstream reaches should be minimized as much as possible to avoid exacerbation of the present shoaling conditions in the tidally influenced portion of the project area. However, capture of wash load materials may not be practical or economically feasible.

Single Event Responses - As described in Chapter 9 for existing conditions, single event simulations of sedimentation dynamics were performed for Alternatives 2 and 3. Accumulated sediment loads at selected locations for Alternatives 2 and 3 for the 2-, 5-, 10-, 25-, 50-, and 100-year flood hydrographs are shown on Figures 10-6 through 10-21. The figures show accumulated sediment loads at the peak of the flood hydrograph (3 days) and at the end of the event (8 days). The values of accumulated load for each event are summarized in Tables 10-2 and 10-3 and Figures 10-12, 10-13 and 10-20, 10-21 for alternatives 2 and 3, respectively. Average annual sediment loads were computed and are also provided in Tables 10-2 and 10-3. The effects of the resultant single event sediment deposition and erosion for the 25-, 50- and 100-year hydrographs on vertical changes in thalweg elevation and instantaneous peak 100-year flood water surface elevation is provided in Tables 10-4 and 10-5. Figures 10-22 and 10-23 provide the longitudinal variation at selected points in the study area of computed instantaneous peak 100-year flood water surface elevation after the passage of the 25-, 50-, and 100-year floods. There is a slight (less than 0.5 feet) reduction in the change in maximum 100-year water surface elevation with Alternatives 2 and 3 compared to existing conditions. Refer to Figures 10-22, 10-23 and 9-16, respectively.

Lower Guadalupe Alternatives for Sediment Evaluation

Reach 1: D/S of Union Pacific Railroad to D/S of Tasman Drive

#	Maintenance Alternative	Maintenance Activities	Expected Capital Improvements
1.1	No Maintenance	1. No maintenance	1. Raise levees by earth fill, flood wall, or other means 2. Modify bridges by replacement or jacking
1.2	Levee Structural Stability and New Vegetation	1. Remove all woody vegetation from levee banks 2. Remove all new woody vegetation from toe to toe of channel bottom	1. Raise levees by earth fill, flood wall, or other means 2. Modify bridges by replacement or jacking
1.3	High-Line	1. Remove all woody vegetation from levee banks 2. Remove all woody vegetation from toe to toe of channel bottom 3. Spray herbicides on channel vegetation from toe to toe outside of 10 meters from either side of low flow channel	1. Raise levees by earth fill, flood wall, or other means
1.4	Restore to Design Conditions	1. Remove all woody vegetation from levee banks 2. Remove channel sediment and/or vegetation to 1983 design conditions	1. Possibly raise levees by earth fill, flood wall, or other means

TABLE 10-1

Lower Guadalupe Alternatives for Sediment Evaluation

Reach 2a: D/S of Tasman Drive to Hetch Hetchy

#	Maintenance Alternative	Maintenance Activities	Expected Capital Improvements
2a.1	No Maintenance	1. No maintenance	1. Raise levees by earth fill, flood wall, or other means 2. Modify bridges by replacement or jacking
2a.2	Levee Structural Stability and New Vegetation	1. Remove all woody vegetation from levee banks 2. Remove all new woody vegetation from toe to toe of channel bottom	1. Raise levees by earth fill, flood wall, or other means 2. Modify bridges by replacement or jacking
2a.3	High-Line	1. Remove all woody vegetation from levee banks 2. Remove all woody vegetation from toe to toe of channel bottom 3. Spray herbicides on channel vegetation from toe to toe outside of 10 meters from either side of low flow channel	1. Raise levees by earth fill, flood wall, or other means
2a.4	Restore to Design Conditions	1. Remove all woody vegetation from levee banks 2. Remove channel sediment and/or vegetation to 1983 design conditions	1. Possibly raise levees by earth fill, flood wall, or other means

TABLE 10-1

Lower Guadalupe Alternatives for Sediment Evaluation

Reach 2b: Hetch Hetchy to D/S of Montague Expressway

#	Maintenance Alternative	Maintenance Activities	Expected Capital Improvements
2b.1	No Maintenance	1. No maintenance	1. Raise levees by earth fill, flood wall, or other means
2b.2	Levee Structural Stability and New Vegetation	1. Remove all woody vegetation from levee banks 2. Remove all new woody vegetation from toe to toe of channel bottom	1. Raise levees by earth fill, flood wall, or other means
2b.3	High-Line/Shaded Riverine Aquatic	1. Remove all woody vegetation from levee banks 2. Remove all woody vegetation from toe to toe outside of 4.6 meters from either side of low flow channel 3. Spray herbicides on channel vegetation from toe to toe outside of 4.6 meters from either side of low flow channel	1. Raise levees by earth fill, flood wall, or other means
2b.4	Restore to Design Conditions	1. Remove all woody vegetation from levee banks 2. Remove channel sediment and/or vegetation to 1983 design conditions	1. Possibly raise levees by earth fill, flood wall, or other means

TABLE 10-1

Lower Guadalupe Alternatives for Sediment Evaluation

Reach 3: D/S of Montague Expressway to D/S of Trimble Road

#	Maintenance Alternative	Maintenance Activities	Expected Capital Improvements
3.1	No Maintenance	1. No maintenance	1. Raise levees by earth fill, flood wall, or other means 2. Modify bridges by replacement or jacking
3.2	Levee Structural Stability and New Vegetation	1. Remove all woody vegetation from levee banks 2. Remove all new woody vegetation from excavated channel	1. Raise levees by earth fill, flood wall, or other means 2. Modify bridges by replacement or jacking
3.3	Sediment Basins	1. Remove all woody vegetation from levee banks 2. Remove all woody vegetation from excavated channel 3. Remove sediment and/or vegetation from one or more constructed sediment basins	1. Raise levees by earth fill, flood wall, or other means 2. Modify bridges by replacement or jacking 3. Construct one or more sediment basins within excavated channel
3.4	Restore to Design Conditions	1. Remove all woody vegetation from levee banks 2. Remove channel sediment and/or vegetation from the excavated channel to 1983 design conditions	1. Possibly raise levees by earth fill, flood wall, or other means

TABLE 10-1

Lower Guadalupe Alternatives for Sediment Evaluation

Reach 4: D/S of Trimble Road to D/S of Highway 101

#	Maintenance Alternative	Maintenance Activities	Expected Capital Improvements
4.1	No Maintenance	1. No maintenance	1. Raise levees by earth fill, flood wall, or other means 2. Modify bridges by replacement or jacking
4.2	Levee Structural Stability and New Vegetation	1. Remove all woody vegetation from levee banks 2. Remove all new woody vegetation from toe to toe of channel bottom	1. Raise levees by earth fill, flood wall, or other means 2. Modify bridges by replacement or jacking
4.3	Shaded Riverine Aquatic	1. Remove all woody vegetation from levee banks 2. Remove all woody vegetation from toe to toe outside of 4.6 meters from either side of low flow channel (This alternative provides SRA by planting woody vegetation within 4.6 meters of low flow channel)	1. Raise levees by earth fill, flood wall, or other means 2. Modify bridges by replacement or jacking
4.4	Restore to Design Conditions	1. Remove all woody vegetation from levee banks 2. Remove channel sediment and/or vegetation to 1983 design conditions	1. Possibly raise levees by earth fill, flood wall, or other means

TABLE 10-1

Lower Guadalupe Alternatives for Sediment Evaluation

Reach A: D/S of Highway 101 to D/S of Interstate 880

#	Maintenance Alternative	Maintenance Activity	Expected Capital Improvements
A.1	No Maintenance	1. No maintenance	1. Raise levees by earth fill, flood wall, or other means 2. Modify bridges by replacement or jacking
A.2	New Vegetation	1. Remove all new woody vegetation from toe to toe of channel bottom	1. Raise levees by earth fill, flood wall, or other means
A.3	Shaded Riverine Aquatic	1. Remove all woody vegetation from toe to toe outside of 4.6 meters from either side of low flow channel (This alternative provides SRA by planting woody vegetation within 4.6 meters of low flow channel, limits on SRA planting will be determined by the Corps required hydraulic grade at I-880)	1. Raise levees by earth fill, flood wall, or other means 2. Modify bridges by replacement or jacking
A.4	Restore to Design Conditions	1. Remove all woody vegetation from channel banks 2. Remove channel sediment and/or vegetation to 1983 design conditions	1. Possibly raise levees by earth fill, flood wall, or other means

TABLE 10-1

Table 10-2. Guadalupe River HEC6t Run (Alternative #2) - tons of sediment passing specific section for various floods

AFTER 3 DAYS (peak flow)

		2 year flood		5 year flood		10 year flood		25 year flood		50 year flood		100 year flood		Average Annual	
Bridge	Location	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)
SPRR	22970	10	7380	19	24084	77	47991	589	92776	1662	144783	2754	167111	102.76	19259.5
Hyw 237	26343	10	7380	22	24084	138	47991	892	92776	2315	144783	3668	167111	143.865	19259.5
Tasman	30255	9	7380	34	24084	250	47991	1361	92776	3280	144783	5001	167111	208.155	19259.5
Montegue	38736	55	7381	297	24084	1238	47991	5419	92776	11261	144783	18564	167111	839.075	19259.8
Trimble	44904	683	7380	4048	24083	9450	47988	20418	92774	33807	144778	46944	167102	3798.49	19258.8
Hyw 101	47578	1310	7377	5321	24080	10449	47938	23193	92764	39206	144766	56896	167083	4662.37	19252.4
I-880	57896	2197	7351	5718	24053	10260	47956	17790	92713	26784	144702	34796	167003	4258.8	19236.5

AFTER 8 DAYS (end of hydrograph)

		2 year flood		5 year flood		10 year flood		25 year flood		50 year flood		100 year flood		Average Annual	
Bridge	Location	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)
SPRR	22970	12	13270	37	45179	100	90156	641	176393	1955	283249	3498	369517	126.435	37076.5
Hyw 237	26343	20	13270	47	45187	163	90156	979	176393	2811	283249	4836	369517	182.305	37078.1
Tasman	30255	52	13270	79	45187	280	90156	1524	176393	4128	283249	6838	369517	279.25	37078.1
Montegue	38736	131	13269	565	45186	1646	90156	6593	176395	15242	283249	25602	369508	1160.36	37077.5
Trimble	44904	1226	13268	6197	45187	13321	90152	29695	176393	52077	283244	76337	369500	5786.89	37076.8
Hyw 101	47578	2139	13264	8555	45184	15654	90145	33729	176383	57492	283233	85052	369481	7092.34	37073.6
I-880	57896	3403	13228	9853	45152	17420	90114	34864	176329	52280	283164	73367	369392	7664.37	37049.4

Table 10-3. Guadalupe River HEC6t Runs - (Alternative #3) - tons of sediment passing specific sections for vaious flood events

AFTER 3 DAYS (peak flow)

		2 year flood		5 year flood		10 year flood		25 year flood		50 year flood		100 year flood		Average Annual	
Bridge	Location	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)
SPRR	22970	15	7381	24	24082	74	47993	553	92774	1566	144777	2679	167122	101.015	19259.6
Hwy 237	26343	15	7381	26	24082	133	47993	841	92774	2185	144777	3583	167122	140.5	19259.6
Tasman	30255	13	7381	37	24082	240	47993	1286	92774	3106	144777	4925	167122	202.405	19259.6
Montegue	38736	90	7381	331	24082	1278	47993	5471	92774	11303	144777	18563	167122	862.27	19259.6
Trimble	44904	699	7381	4127	24081	9622	47990	20682	92772	34027	144771	48828	167116	3874.97	19258.9
Hwy 101	47578	1337	7377	5333	24078	10617	47984	23103	92764	38942	144759	57838	167097	4692.88	19255.8
I-880	57896	2175	7351	5782	24053	10325	47956	17886	92713	27228	144702	34191	167003	4271.63	19236.5

AFTER 8 DAYS (end of hydrograph)

		2 year flood		5 year flood		10 year flood		25 year flood		50 year flood		100 year flood		Average Annual	
Bridge	Location	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)
SPRR	22970	17	13270	47	45190	105	90158	606	176395	1828	283224	3365	369519	125.035	37078.6
Hwy 237	26343	25	13270	61	45190	166	90158	926	176395	2633	283224	4666	369519	179.505	37078.6
Tasman	30255	54	13270	97	45190	278	90158	1442	176395	3876	283224	6638	369519	273.23	37078.6
Montegue	38736	162	13269	752	45189	1890	90158	6760	176393	15131	283224	25545	369519	1230.74	37078
Trimble	44904	1329	13268	6382	45189	13569	90155	30012	176393	51900	283239	79038	369514	5925.17	37077.6
Hwy 101	47578	2159	13264	8508	45185	15838	90147	33701	176384	56874	283226	86503	369492	7115.04	37074.1
I-880	57896	3412	13228	9848	45154	17461	90114	34644	176328	52246	283166	73692	369391	7664.91	37049.8

Table 10-4. Guadalupe River HEC6t Run (Alternative #2) - thalweg & wsp for Q=17,000cfs before and after 8 day event

Elevation Difference in Feet

		before event		after 25 year flood				after 50 year flood				after 100 year flood			
Bridge	Location	before thalweg (feet)	before wsp (feet)	after thalweg (feet)	diff ta-tb (feet)	after wsp (feet)	diff ha-hb (feet)	after thalweg (feet)	diff ta-tb (feet)	after wsp (feet)	diff ha-hb (feet)	after thalweg (feet)	diff ta-tb (feet)	after wsp (feet)	diff ha-hb (feet)
SPRR	22970	-7.48	16.46	-7.48	0	16.46	0	-7.48	0	16.46	0	-7.48	0	16.47	0.01
Hwy 237	26343	0.2	19.76	0.2	0	19.5	-0.26	0.2	0	19.51	-0.25	0.2	0	19.51	-0.25
Tasman	30255	-2.03	23.23	-2.03	0	22.9	-0.33	-2.03	0	22.91	-0.32	-2.03	0	22.92	-0.31
Montegue	38736	2.62	26.48	2.76	0.14	26.56	0.08	2.76	0.14	26.59	0.11	2.76	0.14	26.64	0.16
Trimble	44904	14.34	30.01	13.79	-0.55	29.92	-0.09	13.29	-1.05	30.07	0.06	13.08	-1.26	30.22	0.21
Hwy 101	47578	19.61	32.25	18.13	-1.48	33.3	1.05	17.28	-2.33	33.46	1.21	16.55	-3.06	33.67	1.42
I-880	57896	37.47	56.81	37.77	0.3	56.92	0.11	37.46	-0.01	55.99	-0.82	37.47	0	55.28	-1.53

Elevation Difference in Meters

		before event		after 25 year flood				after 50 year flood				after 100 year flood			
Bridge	Location	before thalweg	before wsp	after thalweg	diff ta-tb	after wsp	diff ha-hb	after thalweg	diff ta-tb	after wsp	diff ha-hb	after thalweg	diff ta-tb	after wsp	diff ha-hb
SPRR	22970	-2.28	5.02	-2.28	0.00	5.02	0.00	-2.28	0.00	5.02	0.00	-2.28	0.00	5.02	0.00
Hwy 237	26343	0.06	6.02	0.06	0.00	5.94	-0.08	0.06	0.00	5.95	-0.08	0.06	0.00	5.95	-0.08
Tasman	30255	-0.62	7.08	-0.62	0.00	6.98	-0.10	-0.62	0.00	6.98	-0.10	-0.62	0.00	6.99	-0.09
Montegue	38736	0.80	8.07	0.84	0.04	8.10	0.02	0.84	0.04	8.10	0.03	0.84	0.04	8.12	0.05
Trimble	44904	4.37	9.15	4.20	-0.17	9.12	-0.03	4.05	-0.32	9.17	0.02	3.99	-0.38	9.21	0.06
Hwy 101	47578	5.98	9.83	5.53	-0.45	10.15	0.32	5.27	-0.71	10.20	0.37	5.04	-0.93	10.26	0.43
I-880	57896	11.42	17.32	11.51	0.09	17.35	0.03	11.42	-0.00	17.07	-0.25	11.42	0.00	16.85	-0.47

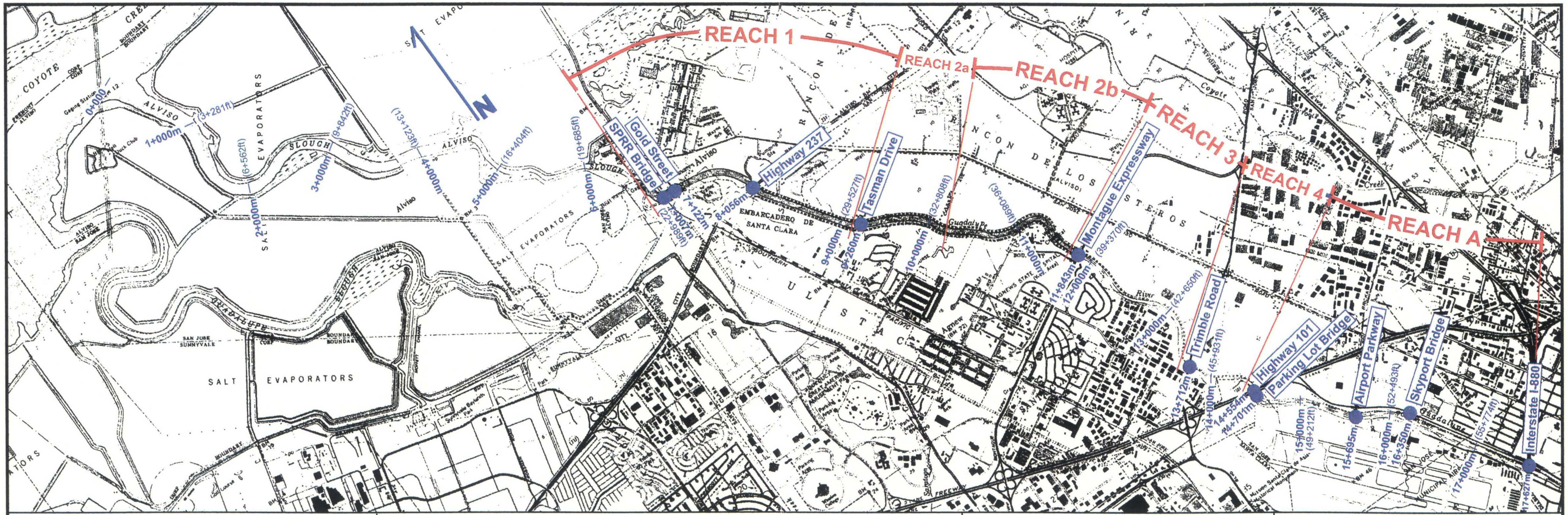
Table 10-5. Guadalupe River HEC6t Run (Alternative #3) - thalweg & wsp for Q=17,000cfs before and after 8 day event

Elevation Difference in Feet

		before event		after 25 year flood				after 50 year flood				after 100 year flood			
Bridge	Location	before thalweg (feet)	before wsp (feet)	after thalweg (feet)	diff ta-tb (feet)	after wsp (feet)	diff ha-hb (feet)	after thalweg (feet)	diff ta-tb (feet)	after wsp (feet)	diff ha-hb (feet)	after thalweg (feet)	diff ta-tb (feet)	after wsp (feet)	diff ha-hb (feet)
SPRR	22970	-7.48	16.46	-7.48	0	16.46	0	-7.48	0	16.46	0	-7.48	0	16.47	0.01
Hwy 237	26343	0.2	19.76	0.2	0	19.3	-0.46	0.2	0	19.3	-0.46	0.2	0	19.3	-0.46
Tasman	30255	-2.03	23.23	-2.03	0	22.62	-0.61	-2.03	0	22.63	-0.6	-2.03	0	22.64	-0.59
Montegue	38736	2.62	26.48	2.76	0.14	26.15	-0.33	2.76	0.14	26.18	-0.3	2.76	0.14	26.34	-0.14
Trimble	44904	14.34	30.01	13.8	-0.54	29.62	-0.39	13.29	-1.05	29.77	-0.24	13.05	-1.29	29.94	-0.07
Hwy 101	47578	19.61	32.25	18.13	-1.48	33.23	0.98	17.29	-2.32	33.35	1.1	16.65	-2.96	33.52	1.27
I-880	57896	37.47	56.81	37.81	0.34	56.73	-0.08	37.47	0	55.94	-0.87	37.47	0	54.99	-1.82

Elevation Difference in Meters

		before event		after 25 year flood				after 50 year flood				after 100 year flood			
Bridge	Location	before thalweg	before wsp	after thalweg	diff ta-tb	after wsp	diff ha-hb	after thalweg	diff ta-tb	after wsp	diff ha-hb	after thalweg	diff ta-tb	after wsp	diff ha-hb
SPRR	22970	-2.28	5.02	-2.28	0.00	5.02	0.00	-2.28	0.00	5.02	0.00	-2.28	0.00	5.02	0.00
Hwy 237	26343	0.06	6.02	0.06	0.00	5.88	-0.14	0.06	0.00	5.88	-0.14	0.06	0.00	5.88	-0.14
Tasman	30255	-0.62	7.08	-0.62	0.00	6.89	-0.19	-0.62	0.00	6.90	-0.18	-0.62	0.00	6.90	-0.18
Montegue	38736	0.80	8.07	0.84	0.04	7.97	-0.10	0.84	0.04	7.98	-0.09	0.84	0.04	8.03	-0.04
Trimble	44904	4.37	9.15	4.21	-0.16	9.03	-0.12	4.05	-0.32	9.07	-0.07	3.98	-0.39	9.13	-0.02
Hwy 101	47578	5.98	9.83	5.53	-0.45	10.13	0.30	5.27	-0.71	10.17	0.34	5.07	-0.90	10.22	0.39
I-880	57896	11.42	17.32	11.52	0.10	17.29	-0.02	11.42	0.00	17.05	-0.27	11.42	0.00	16.76	-0.55



northwest hydraulic consultants

notes:
- base mapping from USGS quadrangle sheets San Jose West, CA; Milpitas, CA; and Mountain View, CA.
- All stationing is in meters, with stationing in feet shown in parentheses using the conversion 1 m = 3.2808
- Stationing indicating bridge locations is approximate.

0 1 2
kilometers

Figure 10-1. Subreaches of project alternatives.

nhc

Figure 10-2 Water Surf. Elev. Change

Alternative 2

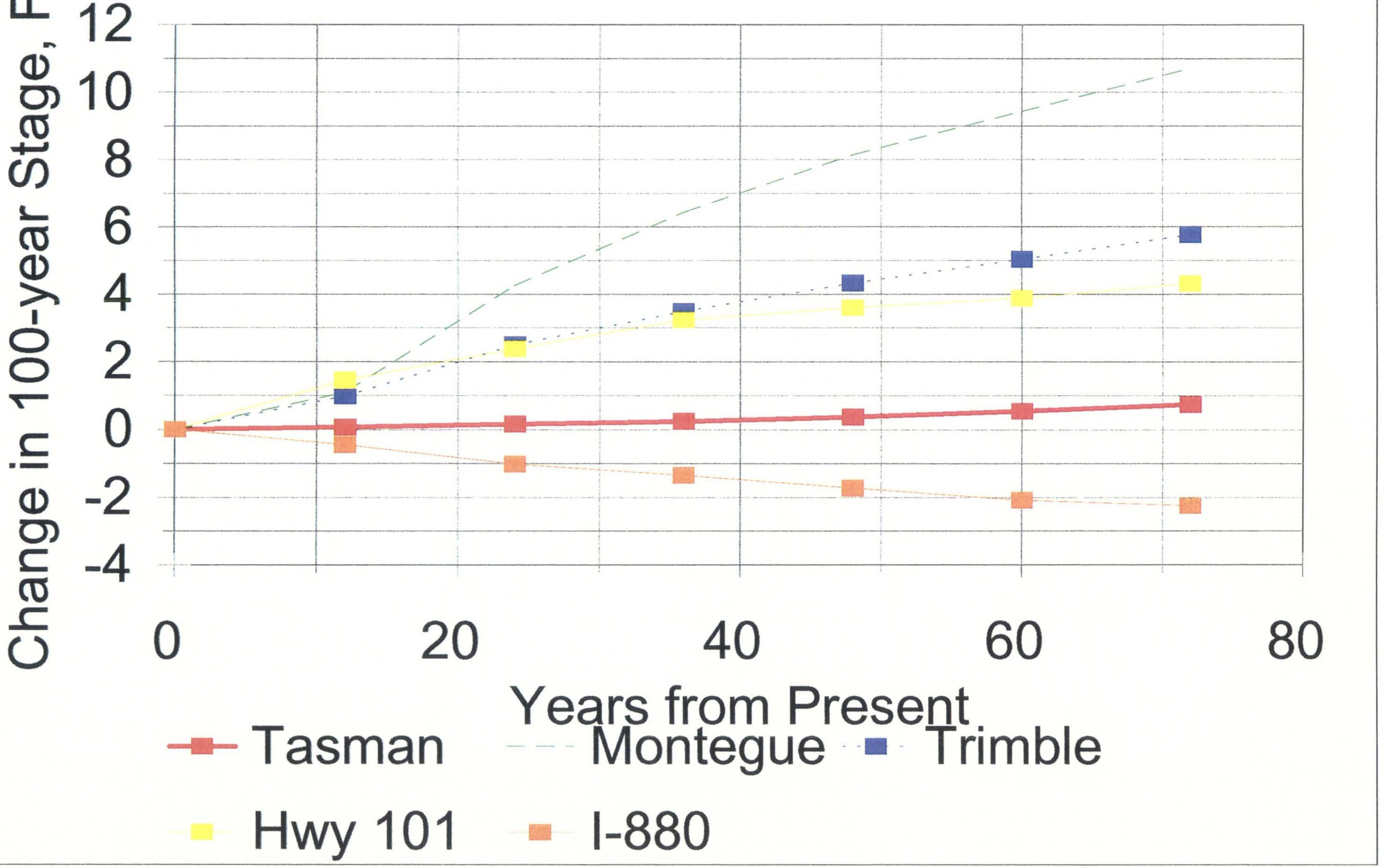


Figure 10-3 Water Surf. Elev. Change

Alternative 2

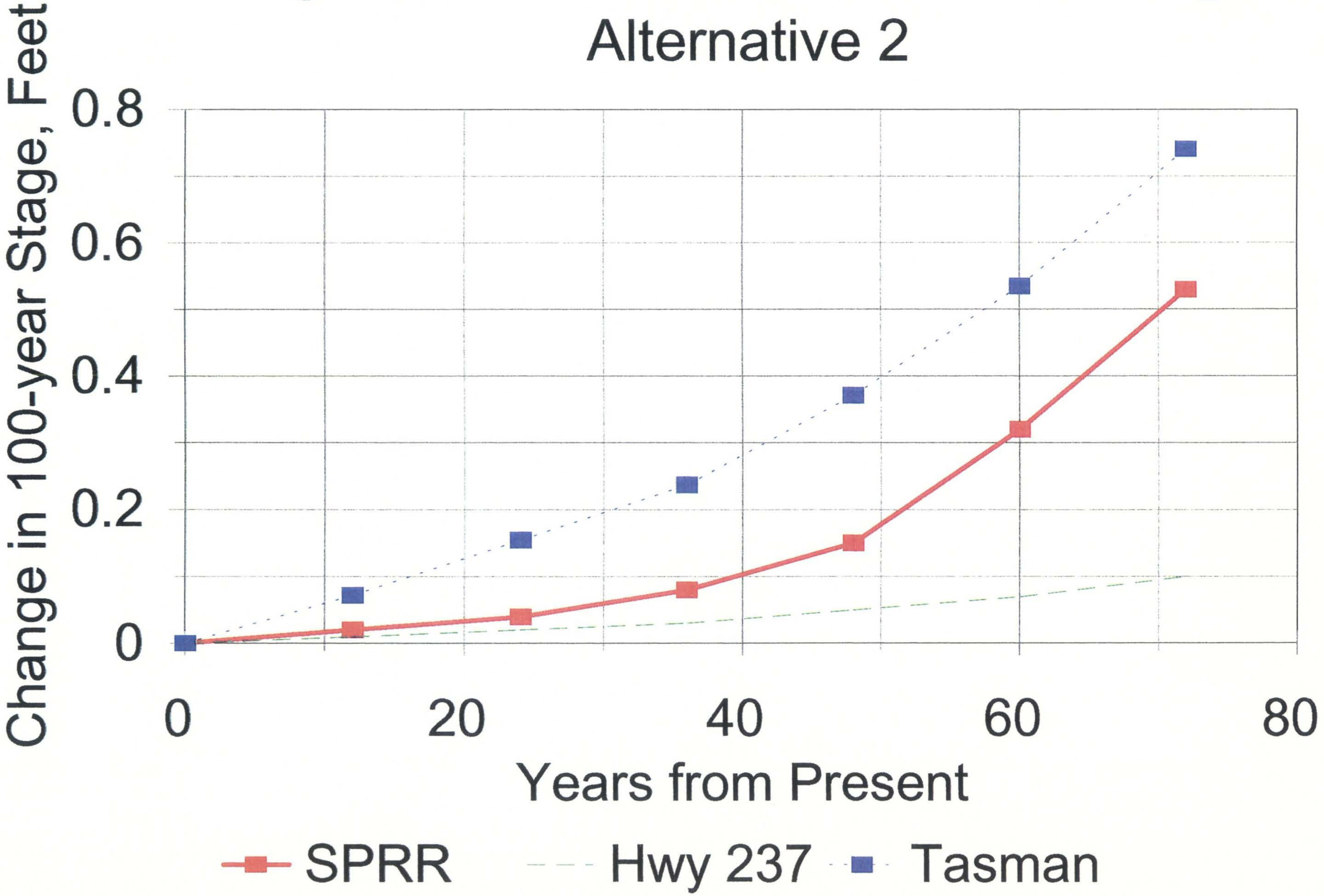
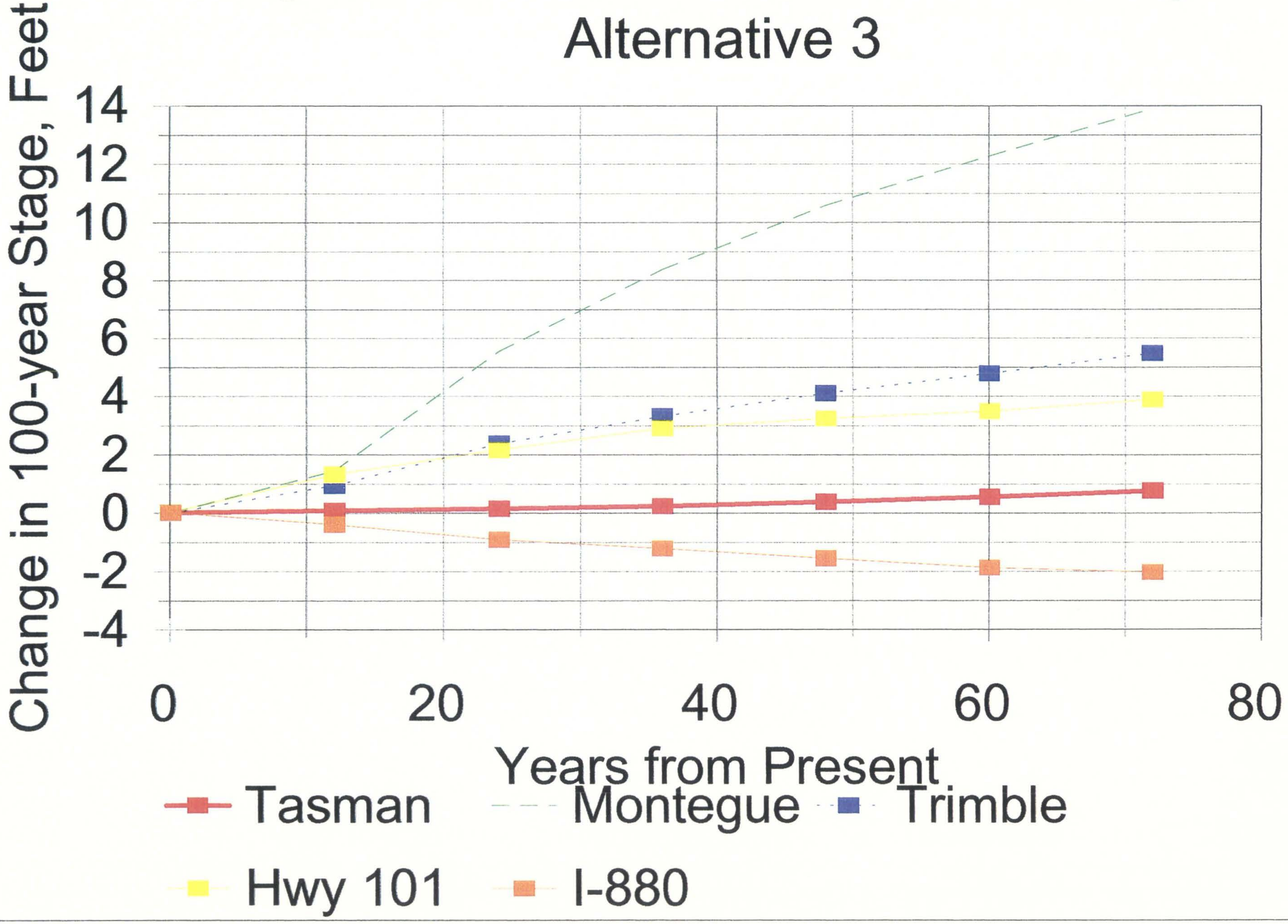


Figure 10-4 Water Surf. Elev. Change

Alternative 3



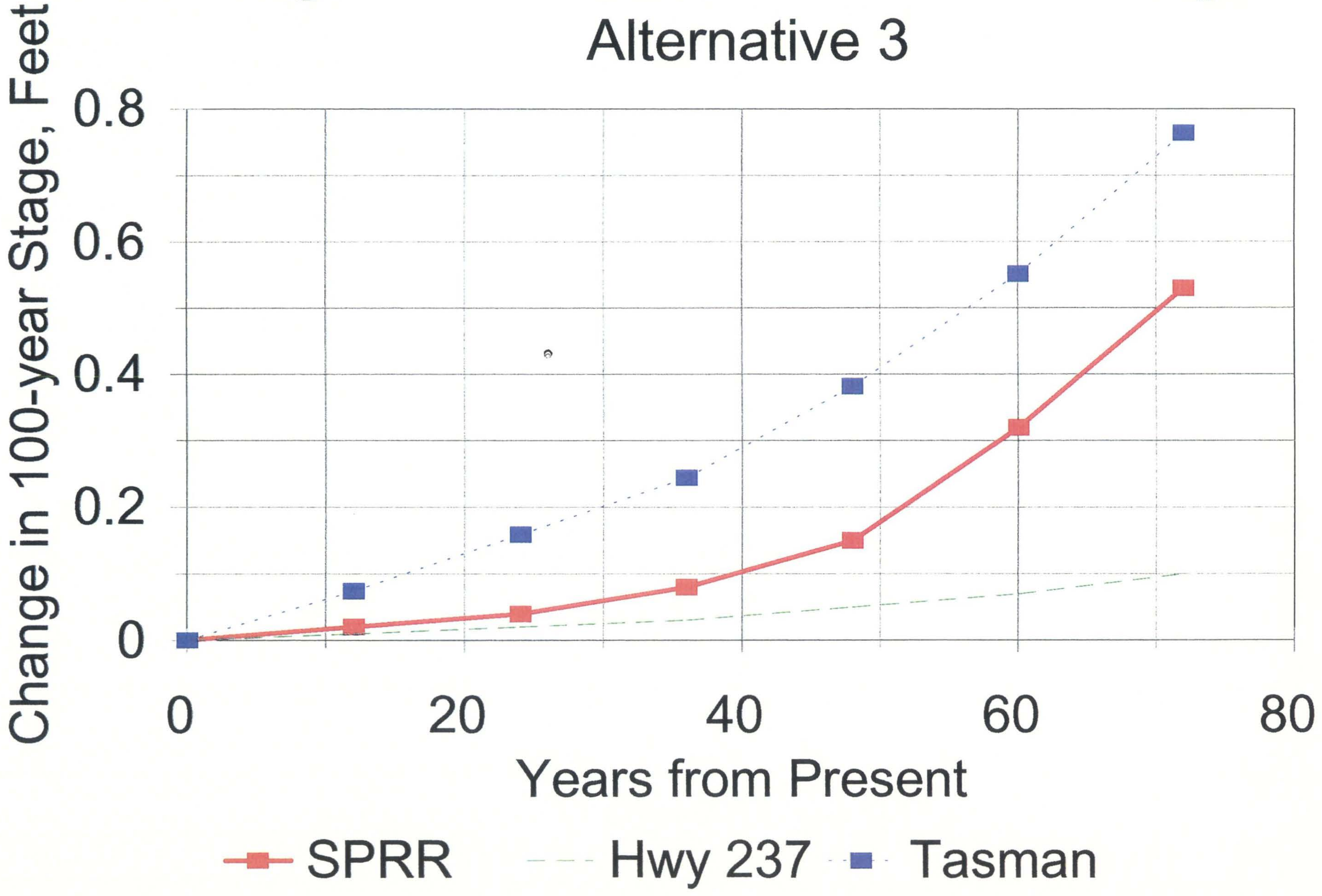


Figure 10-5 Water Surf. Elev. Change

Alternative 3

Sediment Delivery

2-year flood (alternative #2)

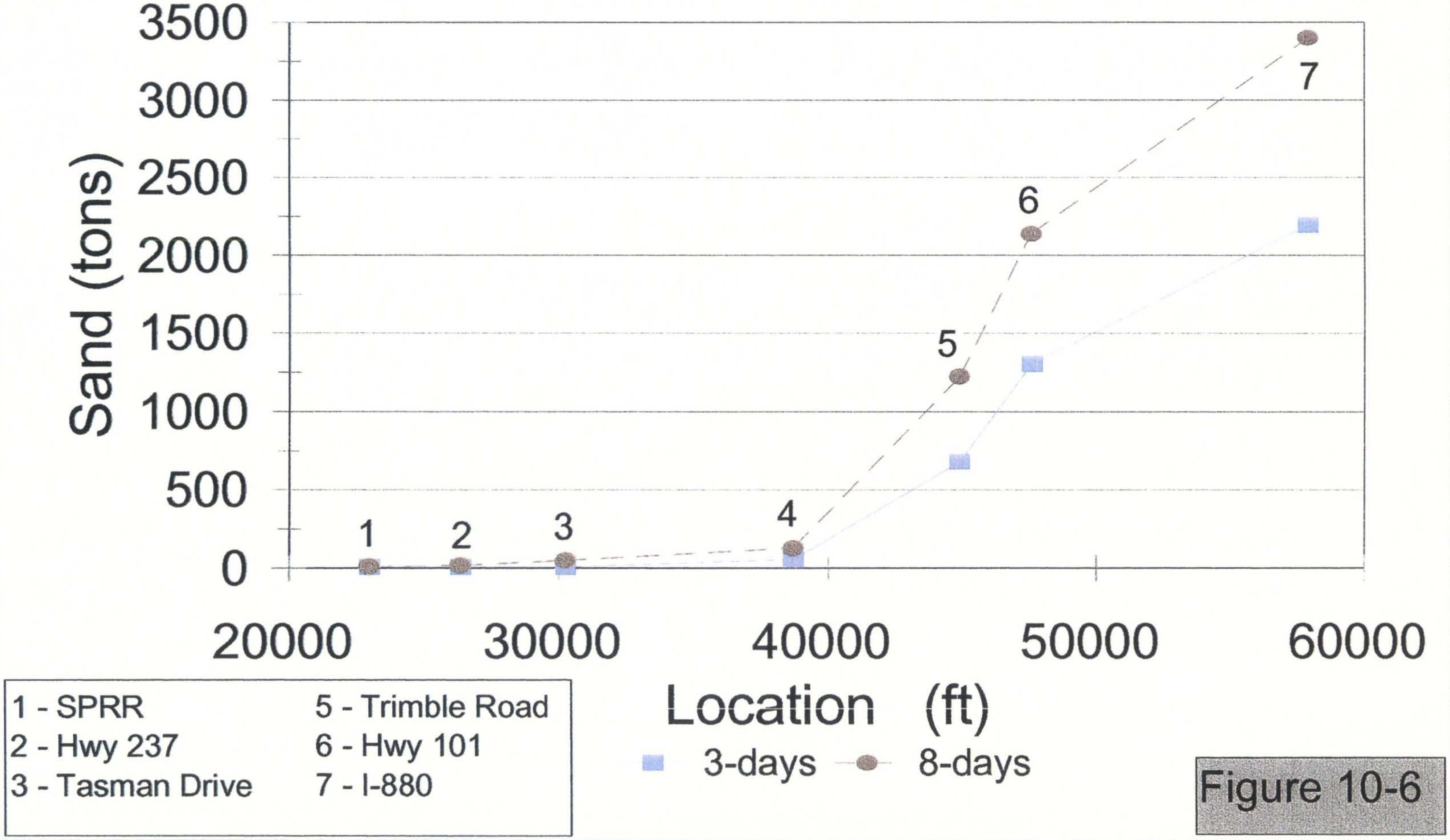
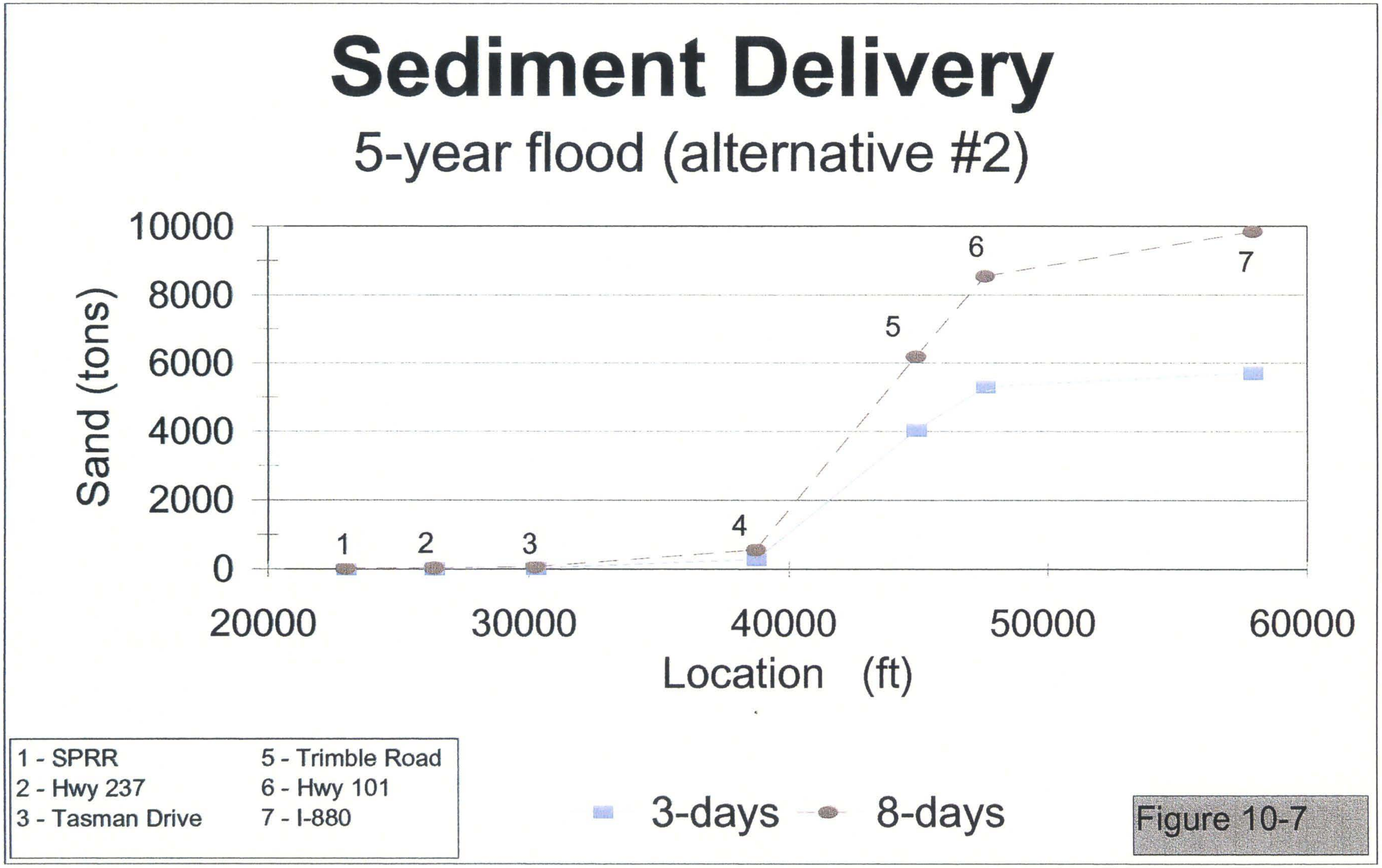


Figure 10-6



Sediment Delivery

5-year flood (alternative #2)

Figure 10-7

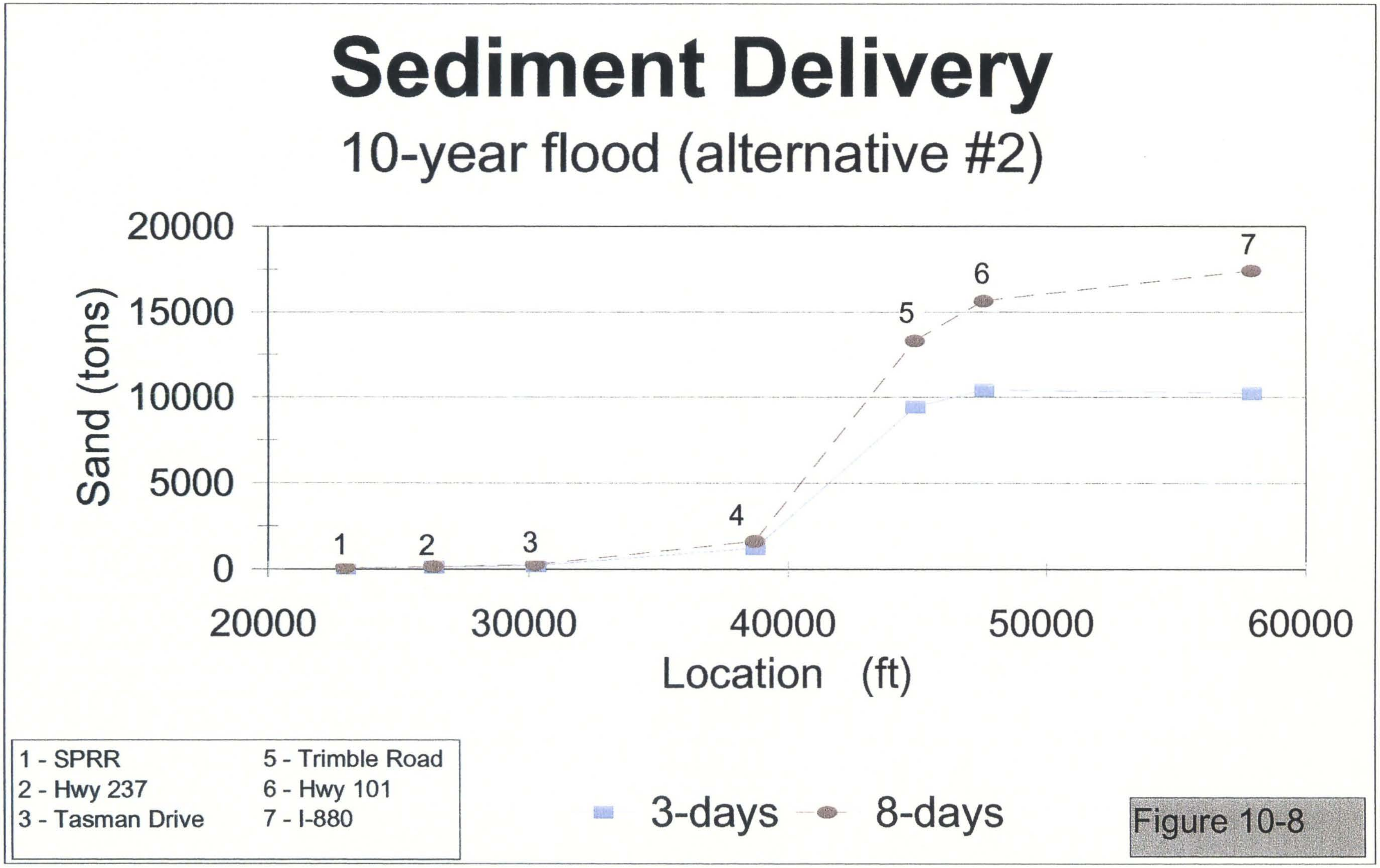

Sediment Delivery
10-year flood (alternative #2)
Sand (tons)
20000
15000
10000
5000
0
20000
30000
40000
50000
60000
Location (ft)
1
2
3
4
5
6
7
1 - SPRR
2 - Hwy 237
3 - Tasman Drive
5 - Trimble Road
6 - Hwy 101
7 - I-880
3-days
8-days

Figure 10-8

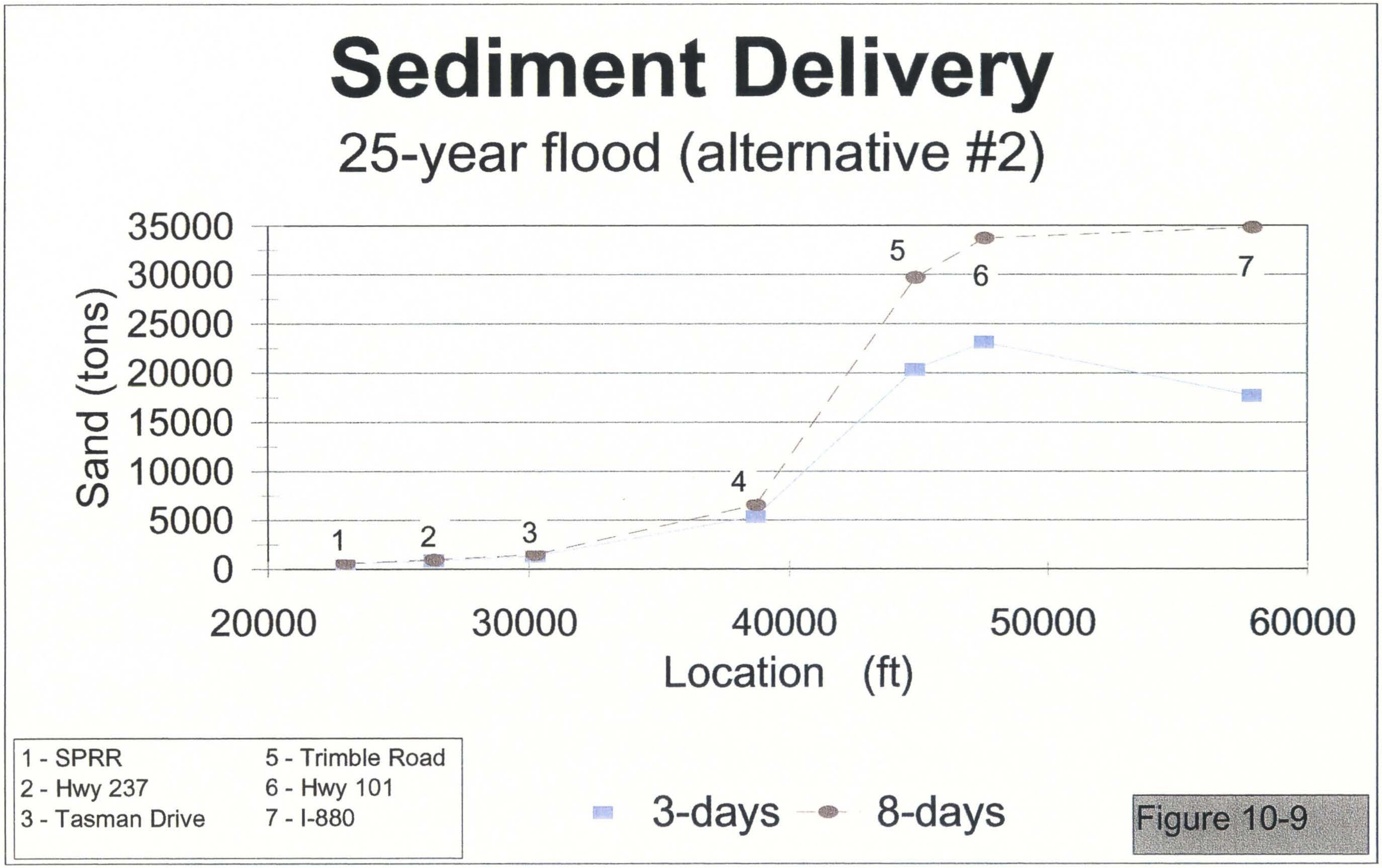

Sediment Delivery
25-year flood (alternative #2)
Sand (tons)
35000
30000
25000
20000
15000
10000
5000
0
20000
30000
40000
50000
60000
Location (ft)
1
2
3
4
5
6
7
1 - SPRR
2 - Hwy 237
3 - Tasman Drive
5 - Trimble Road
6 - Hwy 101
7 - I-880
3-days
8-days

Figure 10-9

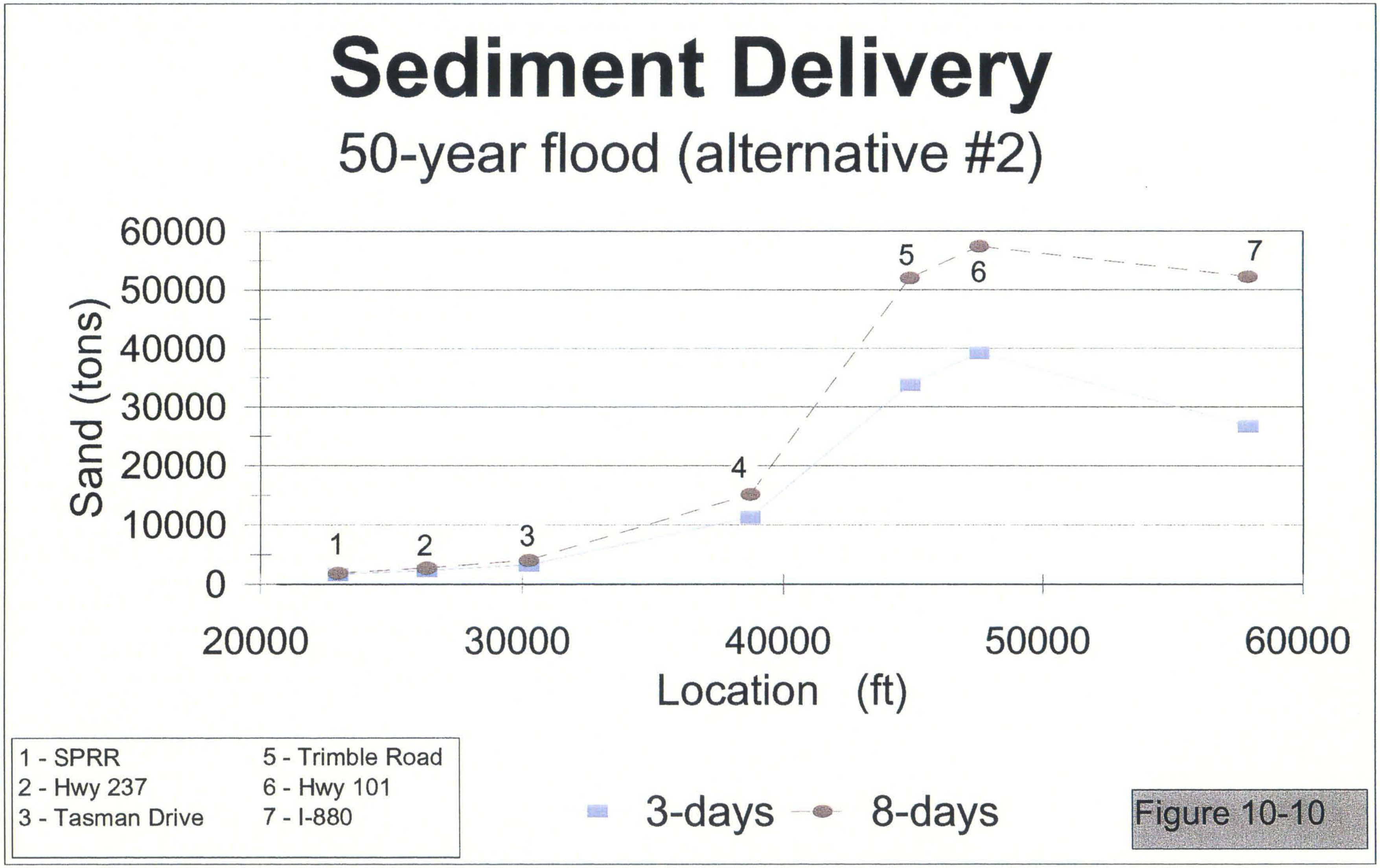
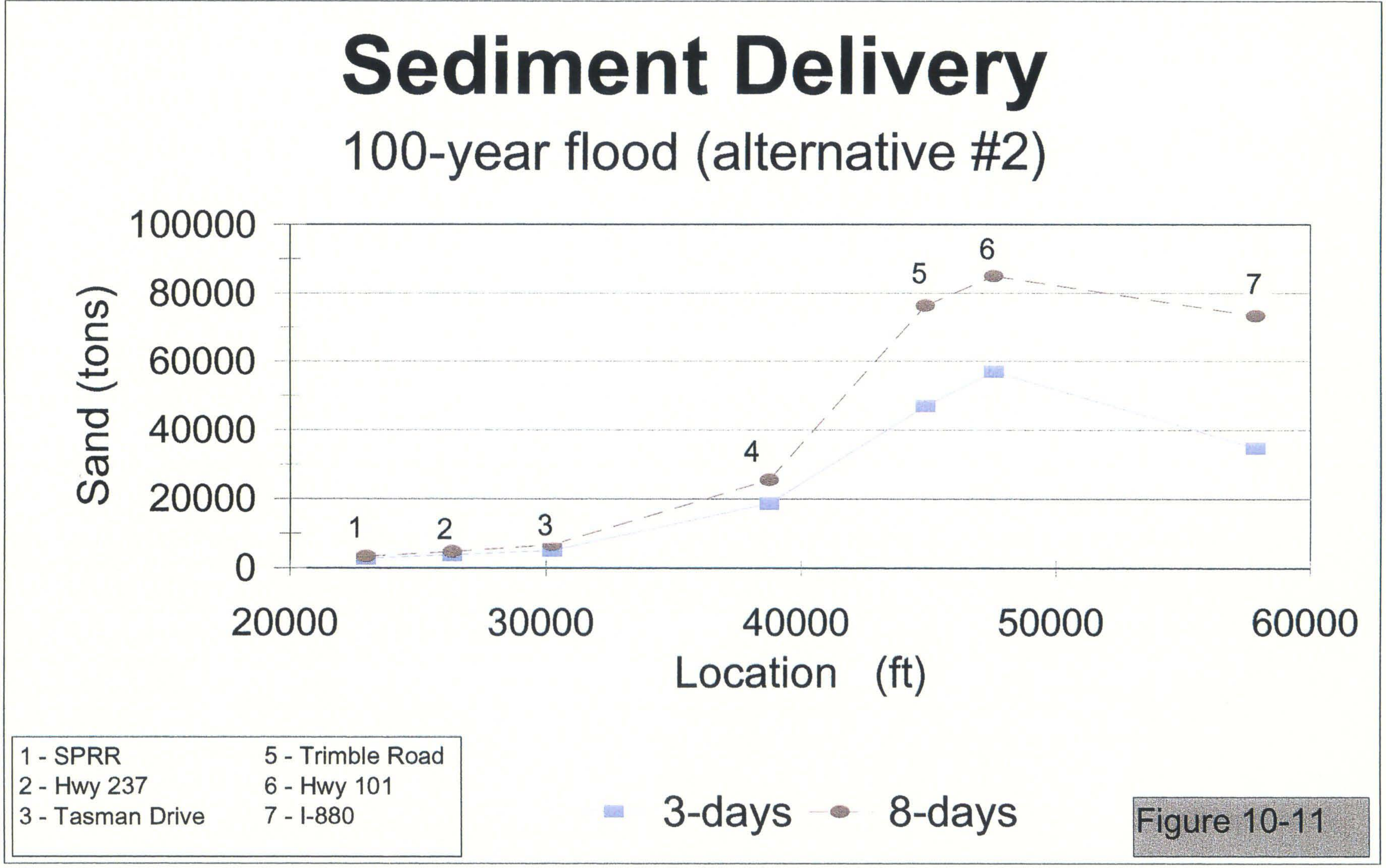

Sediment Delivery
50-year flood (alternative #2)
Sand (tons)
60000
50000
40000
30000
20000
10000
0
20000
30000
40000
50000
60000
Location (ft)
1
2
3
4
5
6
7
1 - SPRR
2 - Hwy 237
3 - Tasman Drive
5 - Trimble Road
6 - Hwy 101
7 - I-880
3-days
8-days

Figure 10-10



Sediment Delivery

100-year flood (alternative #2)

Figure 10-11

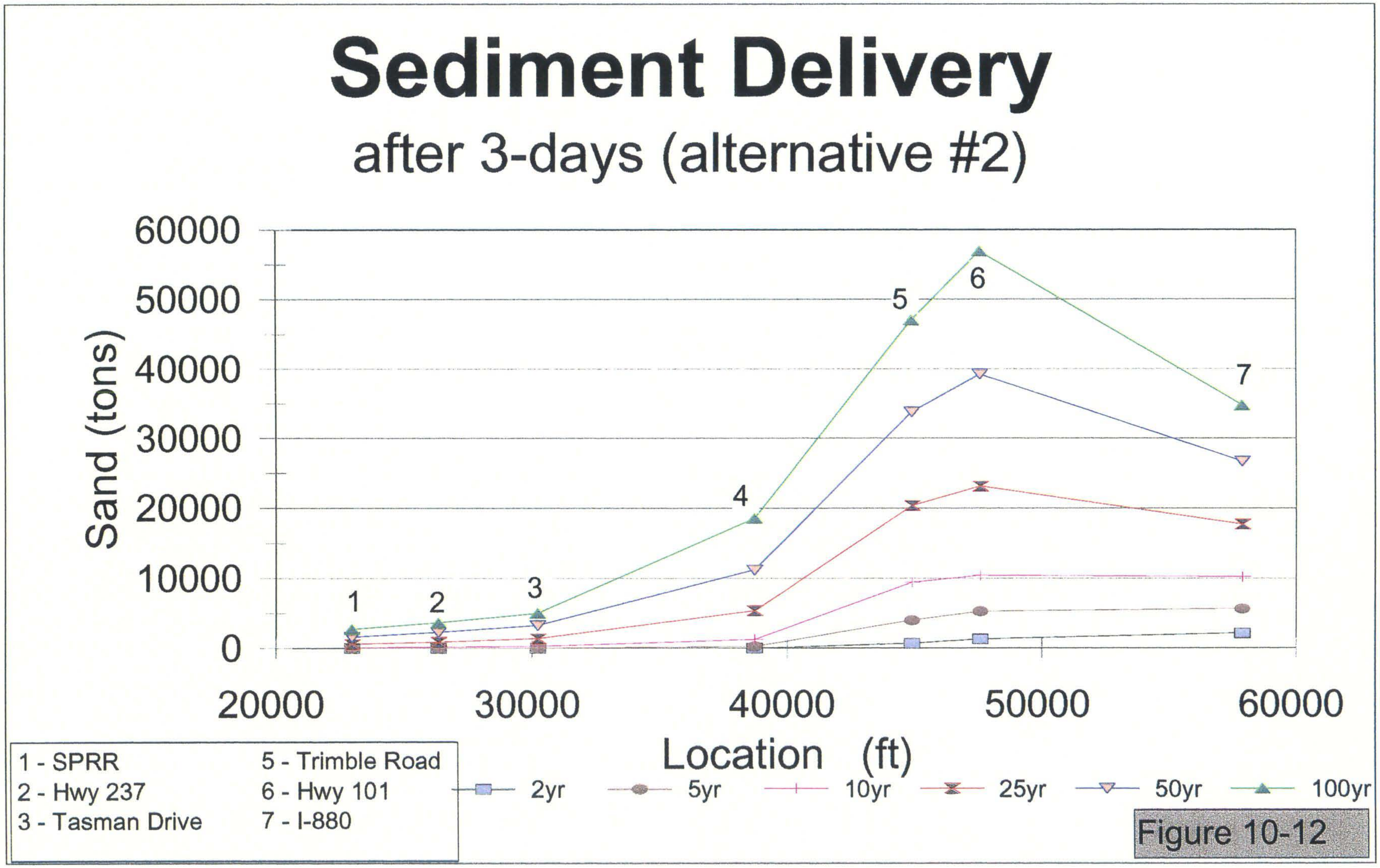

Sediment Delivery
after 3-days (alternative #2)
Sand (tons)
60000
50000
40000
30000
20000
10000
0
20000
30000
40000
50000
60000
Location (ft)
1
2
3
4
5
6
7
1 - SPRR
2 - Hwy 237
3 - Tasman Drive
5 - Trimble Road
6 - Hwy 101
7 - I-880
2yr
5yr
10yr
25yr
50yr
100yr

Figure 10-12

Sediment Delivery

after 8-days (alternative #2)

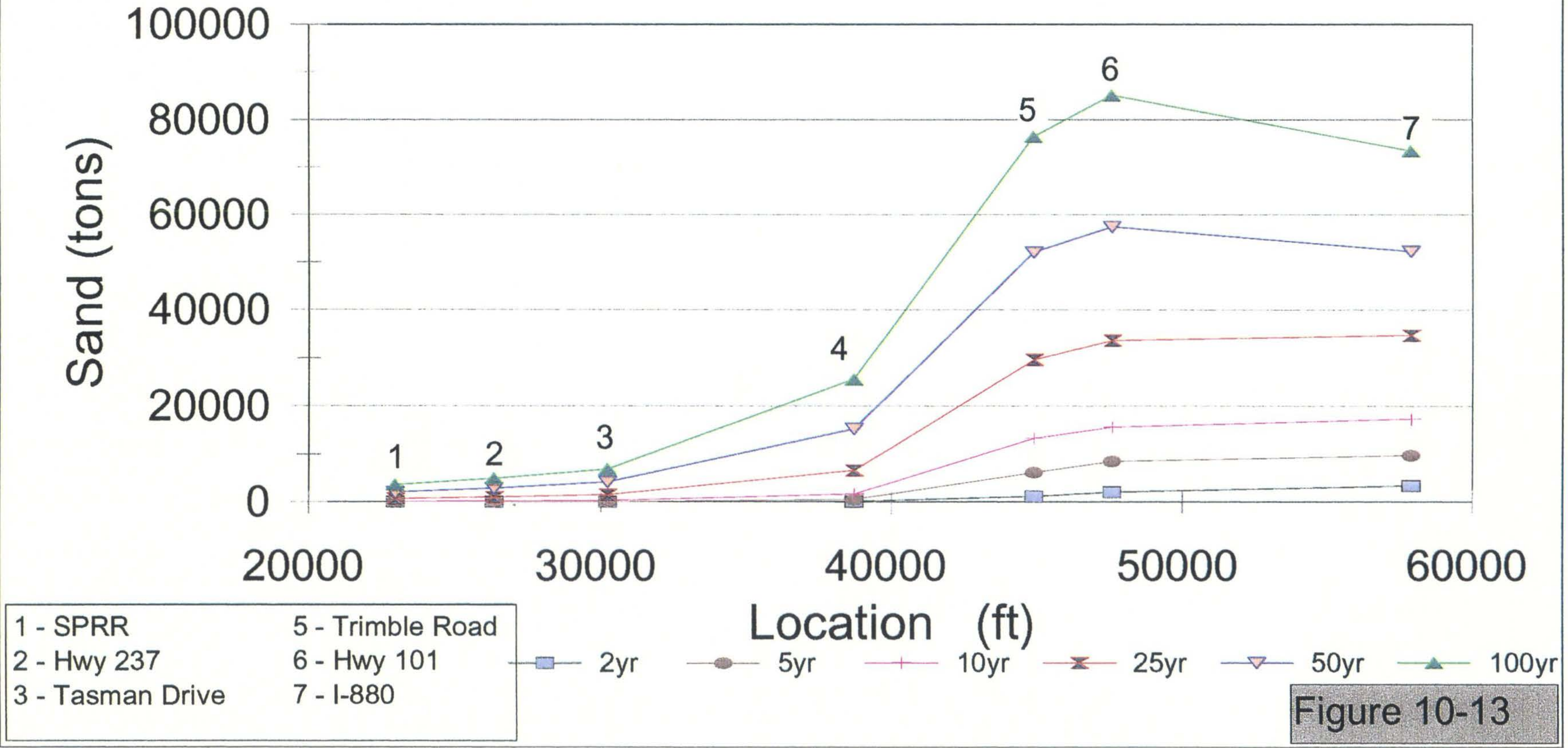


Figure 10-13

Sediment Delivery

2-year flood (alternative #3)

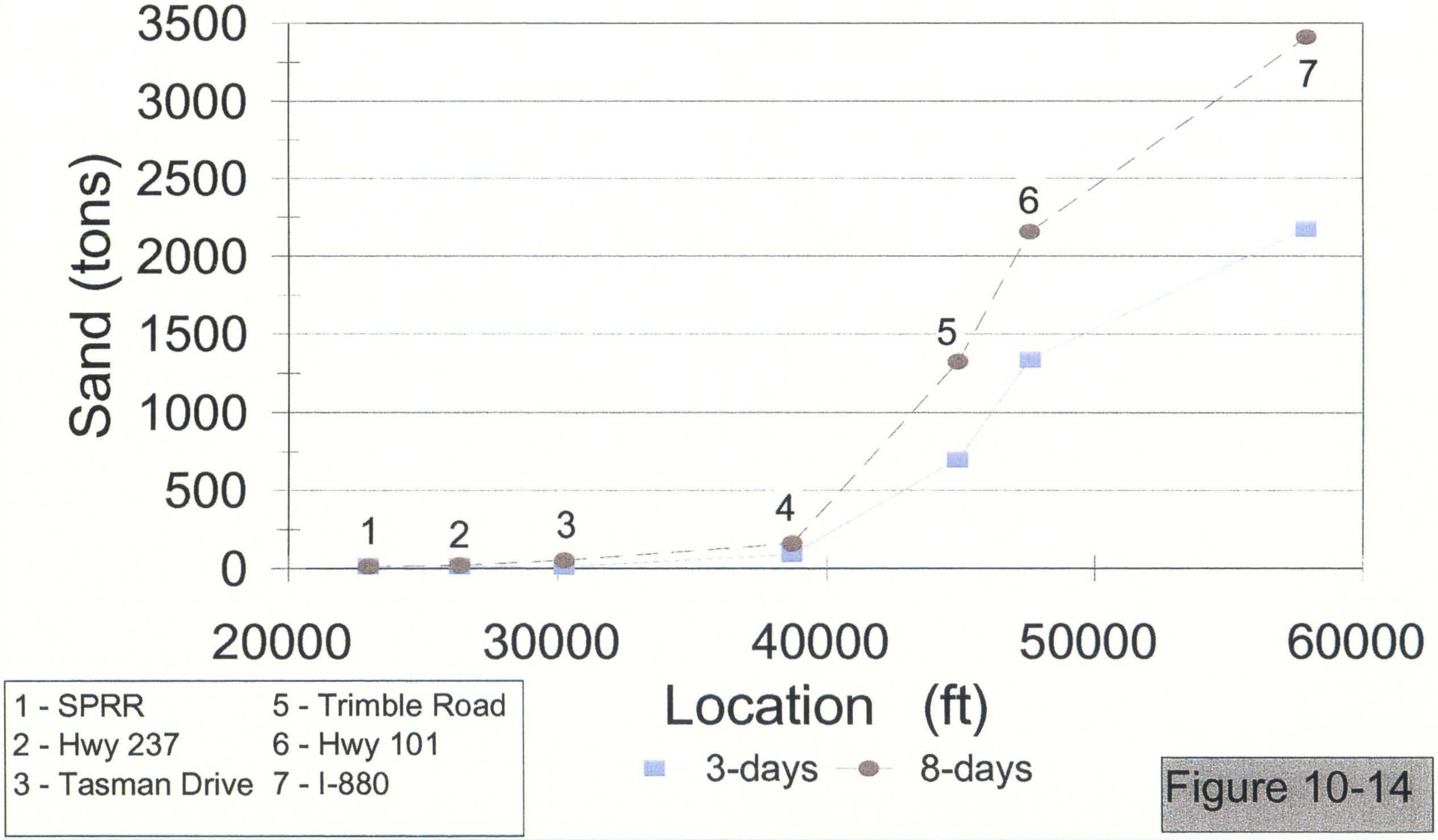


Figure 10-14

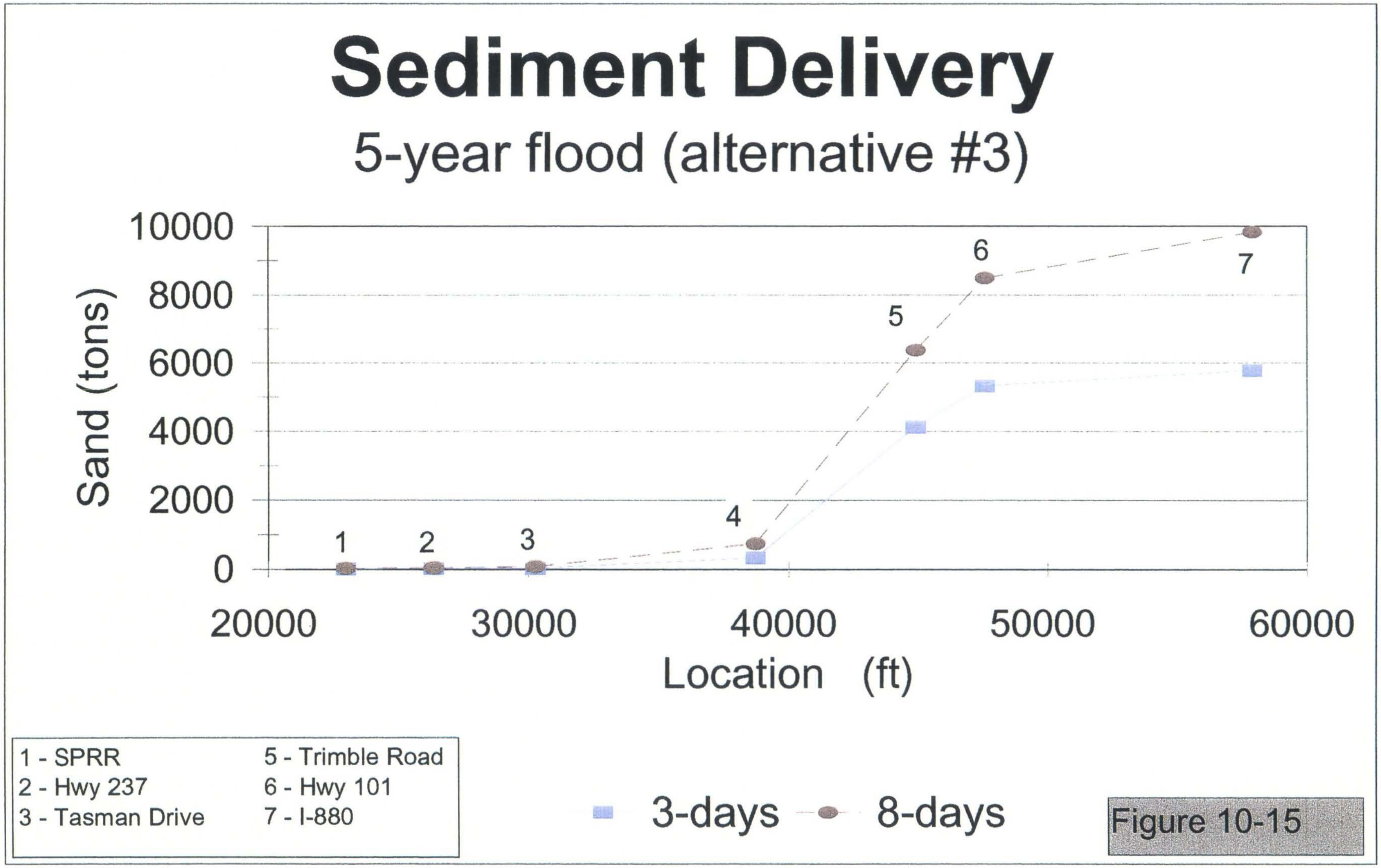

Sediment Delivery
5-year flood (alternative #3)
Sand (tons)
10000
8000
6000
4000
2000
0
20000
30000
40000
50000
60000
Location (ft)
1
2
3
4
5
6
7
1 - SPRR
2 - Hwy 237
3 - Tasman Drive
5 - Trimble Road
6 - Hwy 101
7 - I-880
3-days
8-days

Figure 10-15

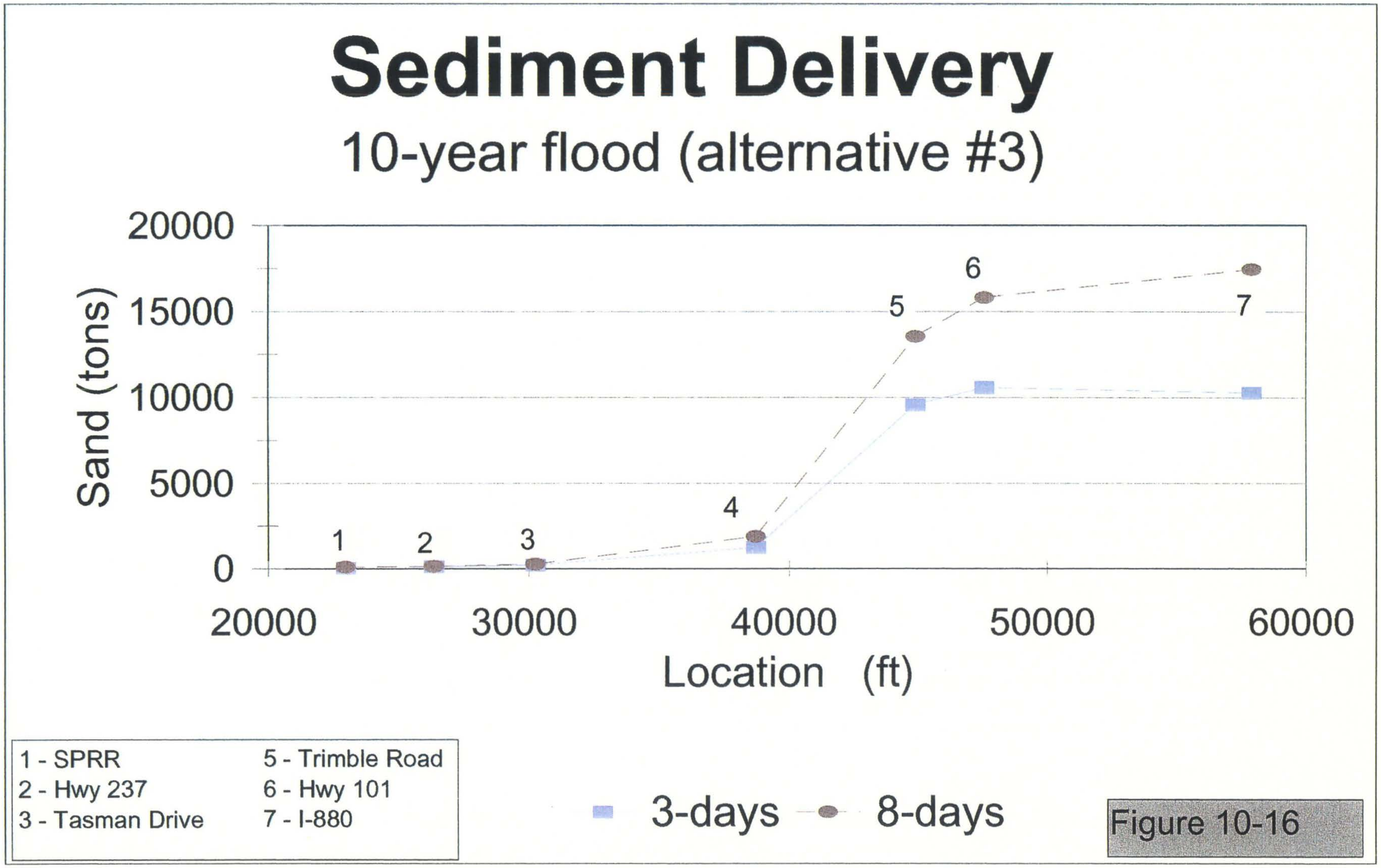

Sediment Delivery
10-year flood (alternative #3)
Sand (tons)
20000
15000
10000
5000
0
20000
30000
40000
50000
60000
Location (ft)
1
2
3
4
5
6
7
1 - SPRR
2 - Hwy 237
3 - Tasman Drive
5 - Trimble Road
6 - Hwy 101
7 - I-880
3-days
8-days

Figure 10-16

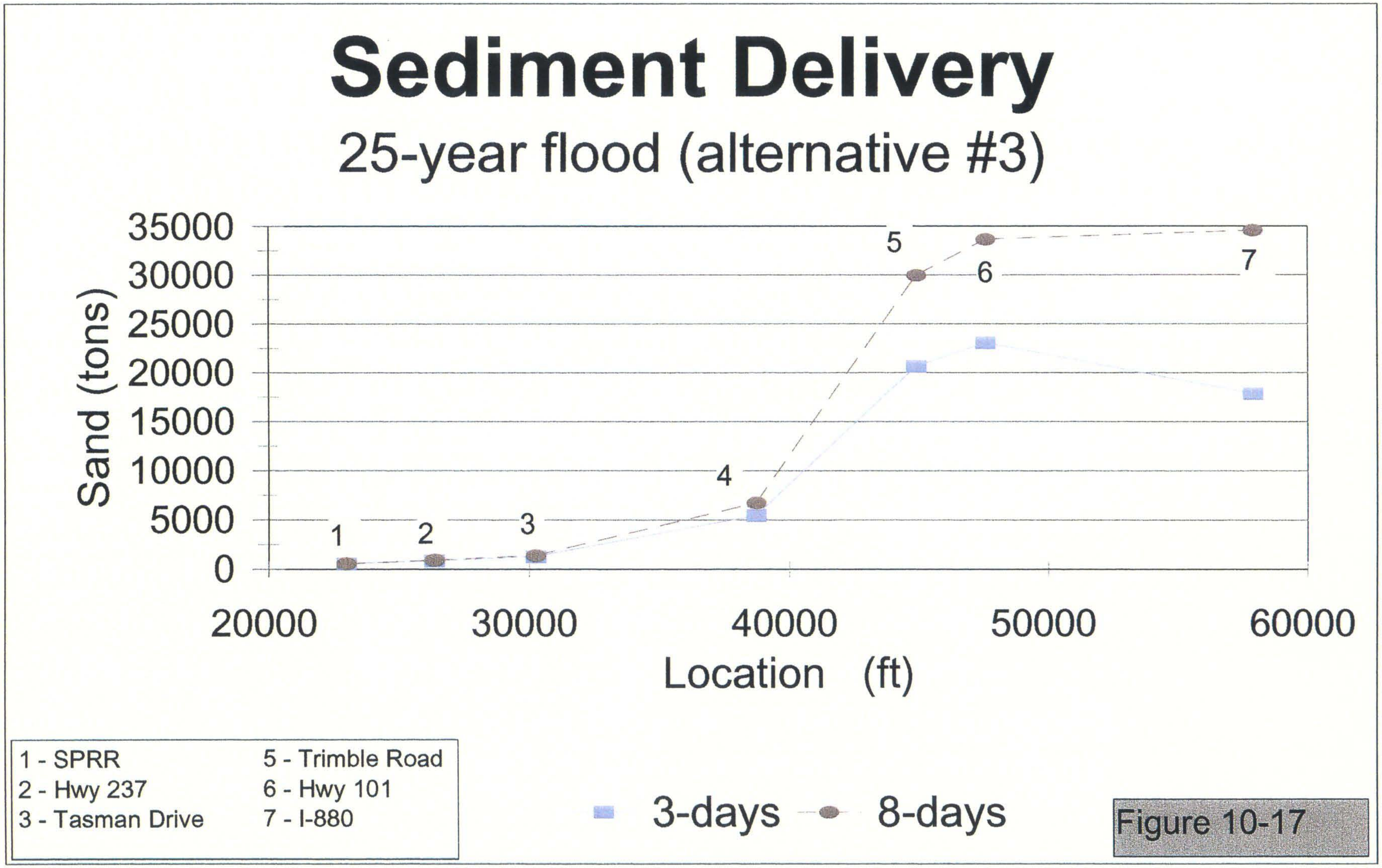

Sediment Delivery
25-year flood (alternative #3)
Sand (tons)
35000
30000
25000
20000
15000
10000
5000
0
20000
30000
40000
50000
60000
Location (ft)
1
2
3
4
5
6
7
1 - SPRR
2 - Hwy 237
3 - Tasman Drive
5 - Trimble Road
6 - Hwy 101
7 - I-880
3-days
8-days

Figure 10-17

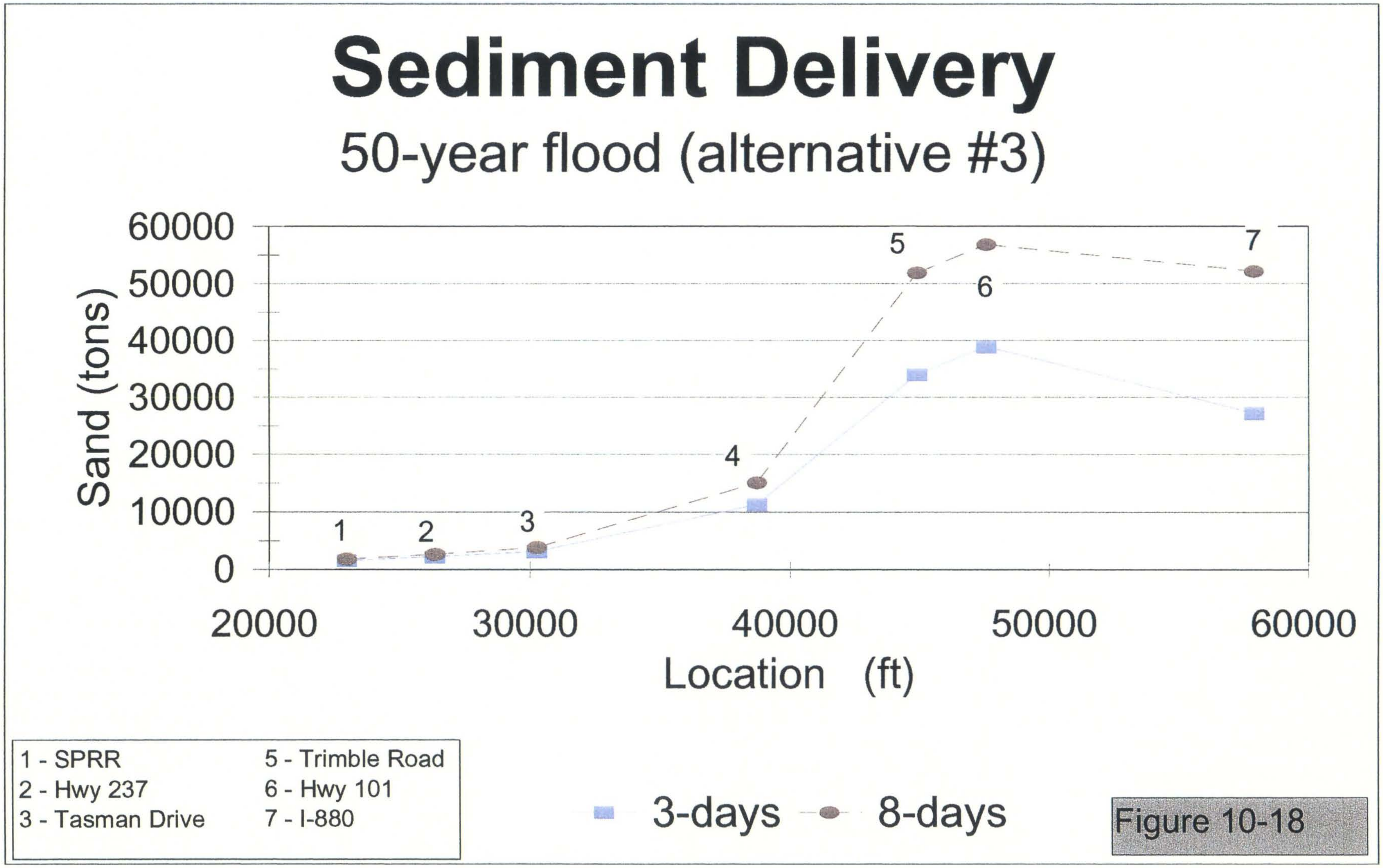
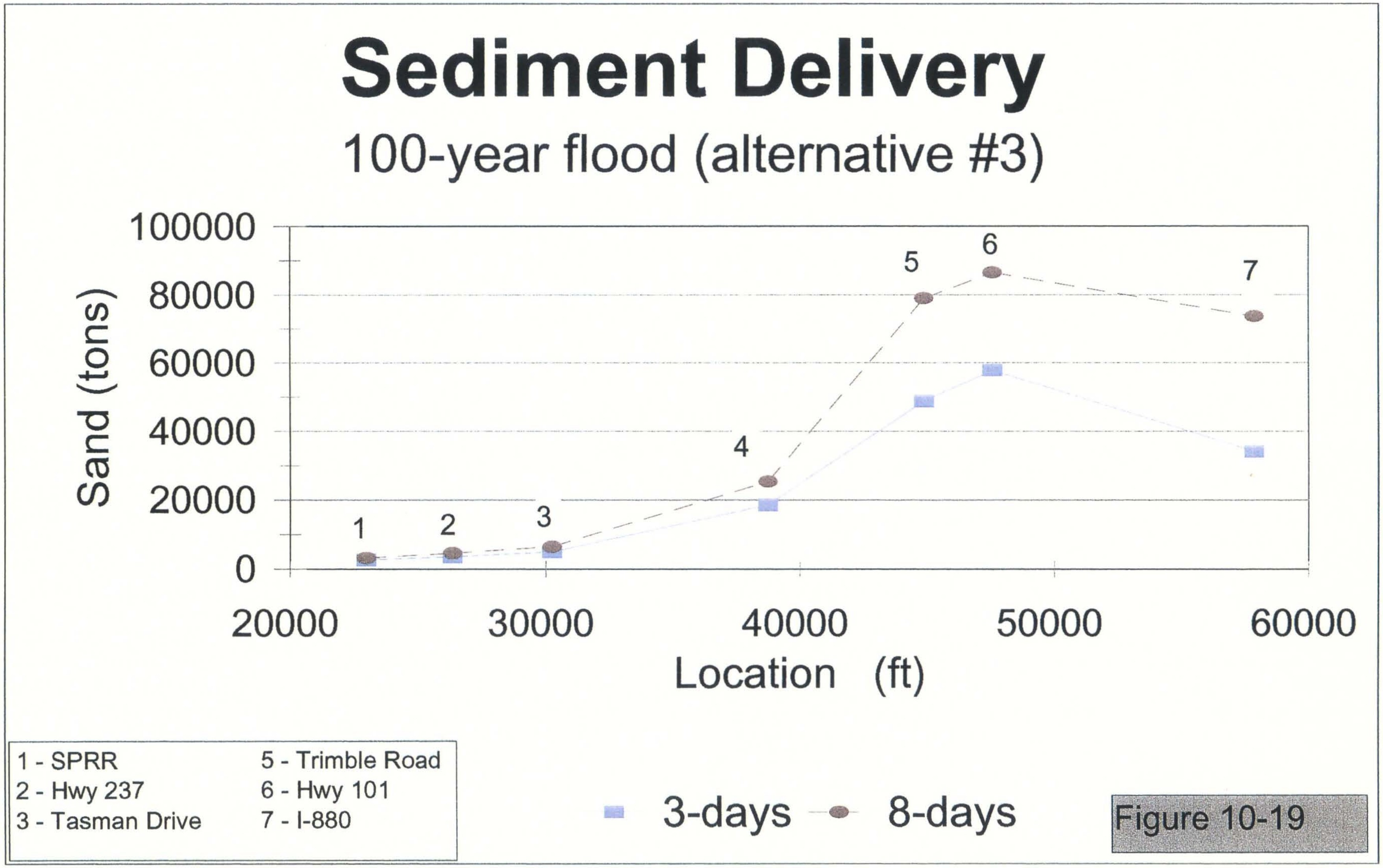

Sediment Delivery
50-year flood (alternative #3)
Sand (tons)
60000
50000
40000
30000
20000
10000
0
20000
30000
40000
50000
60000
Location (ft)
1
2
3
4
5
6
7
1 - SPRR
2 - Hwy 237
3 - Tasman Drive
5 - Trimble Road
6 - Hwy 101
7 - I-880
3-days
8-days

Figure 10-18



Sediment Delivery
100-year flood (alternative #3)
Sand (tons)
100000
80000
60000
40000
20000
0
20000
30000
40000
50000
60000
Location (ft)
1
2
3
4
5
6
7
1 - SPRR
2 - Hwy 237
3 - Tasman Drive
5 - Trimble Road
6 - Hwy 101
7 - I-880
3-days
8-days

Figure 10-19

Sediment Delivery

after 3-days (alternative #3)

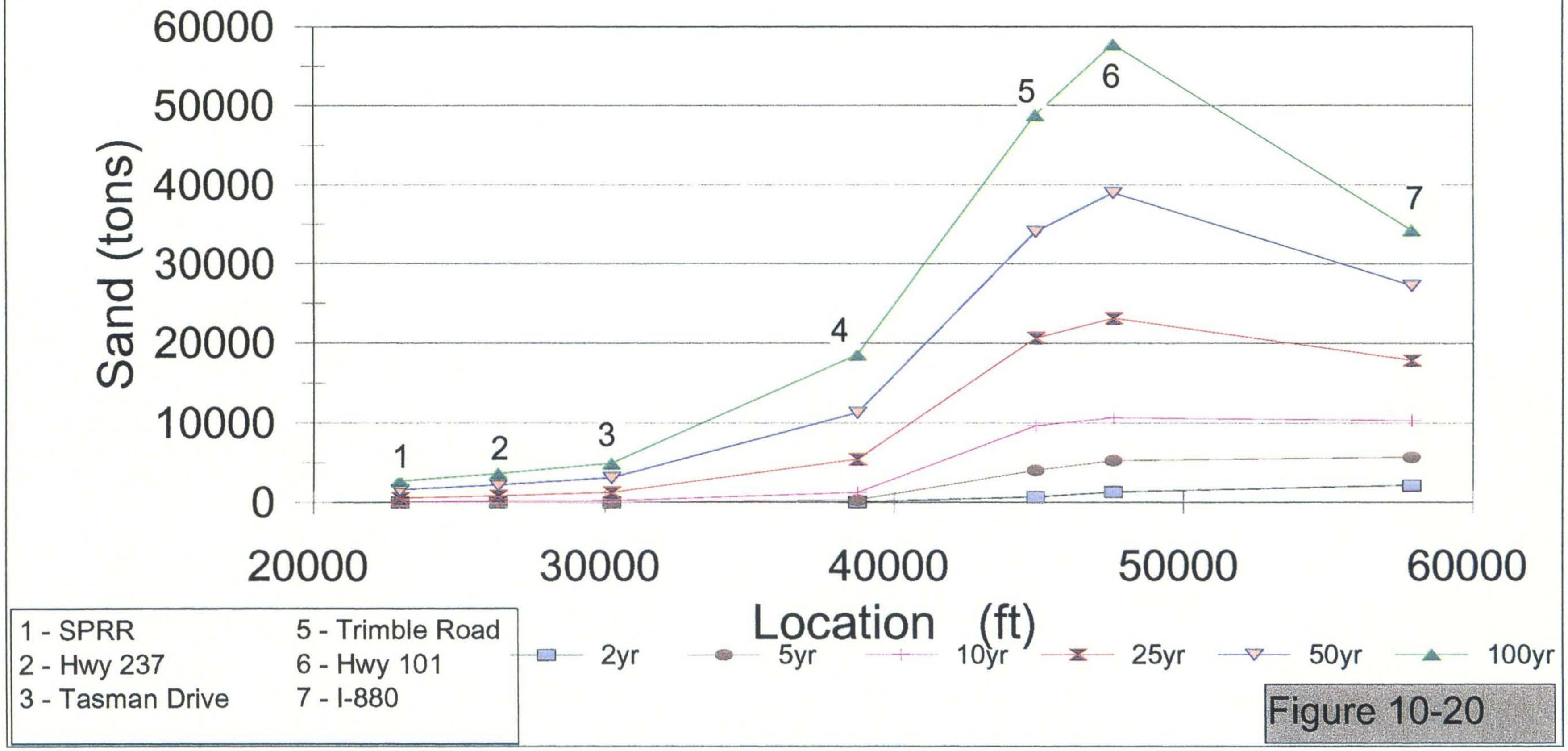


Figure 10-20

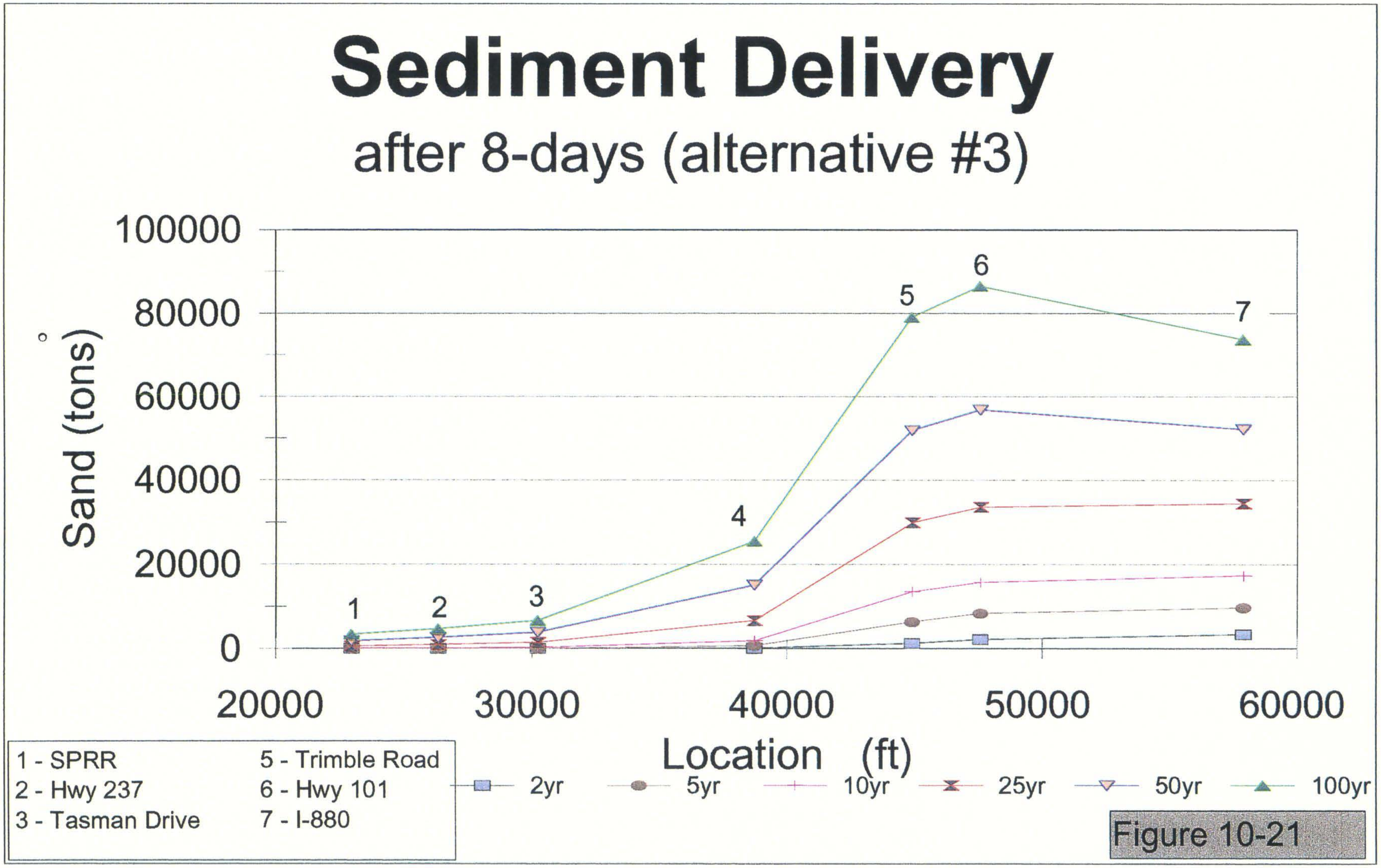


Figure 10-21

Sediment Delivery
after 8-days (alternative #3)
Sand (tons)
100000
80000
60000
40000
20000
0
20000
30000
40000
50000
60000
Location (ft)
1
2
3
4
5
6
7
1 - SPRR
2 - Hwy 237
3 - Tasman Drive
5 - Trimble Road
6 - Hwy 101
7 - I-880
2yr
5yr
10yr
25yr
50yr
100yr

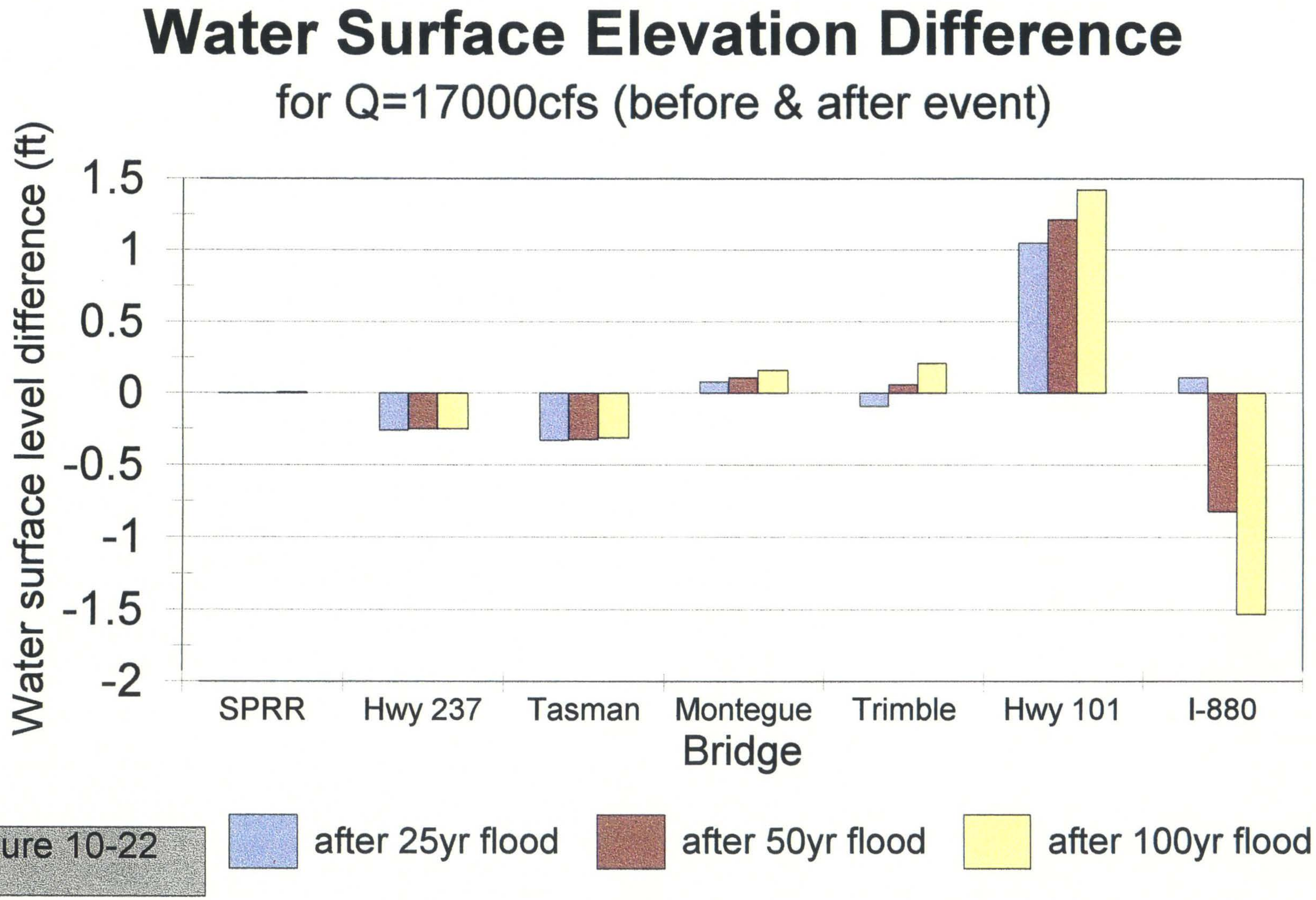

Water Surface Elevation Difference
for Q=17000cfs (before & after event)
Water surface level difference (ft)
1.5
1
0.5
0
-0.5
-1
-1.5
-2
SPRR
Hwy 237
Tasman
Montegue
Trimble
Hwy 101
I-880
Bridge
after 25yr flood
after 50yr flood
after 100yr flood

Figure 10-22

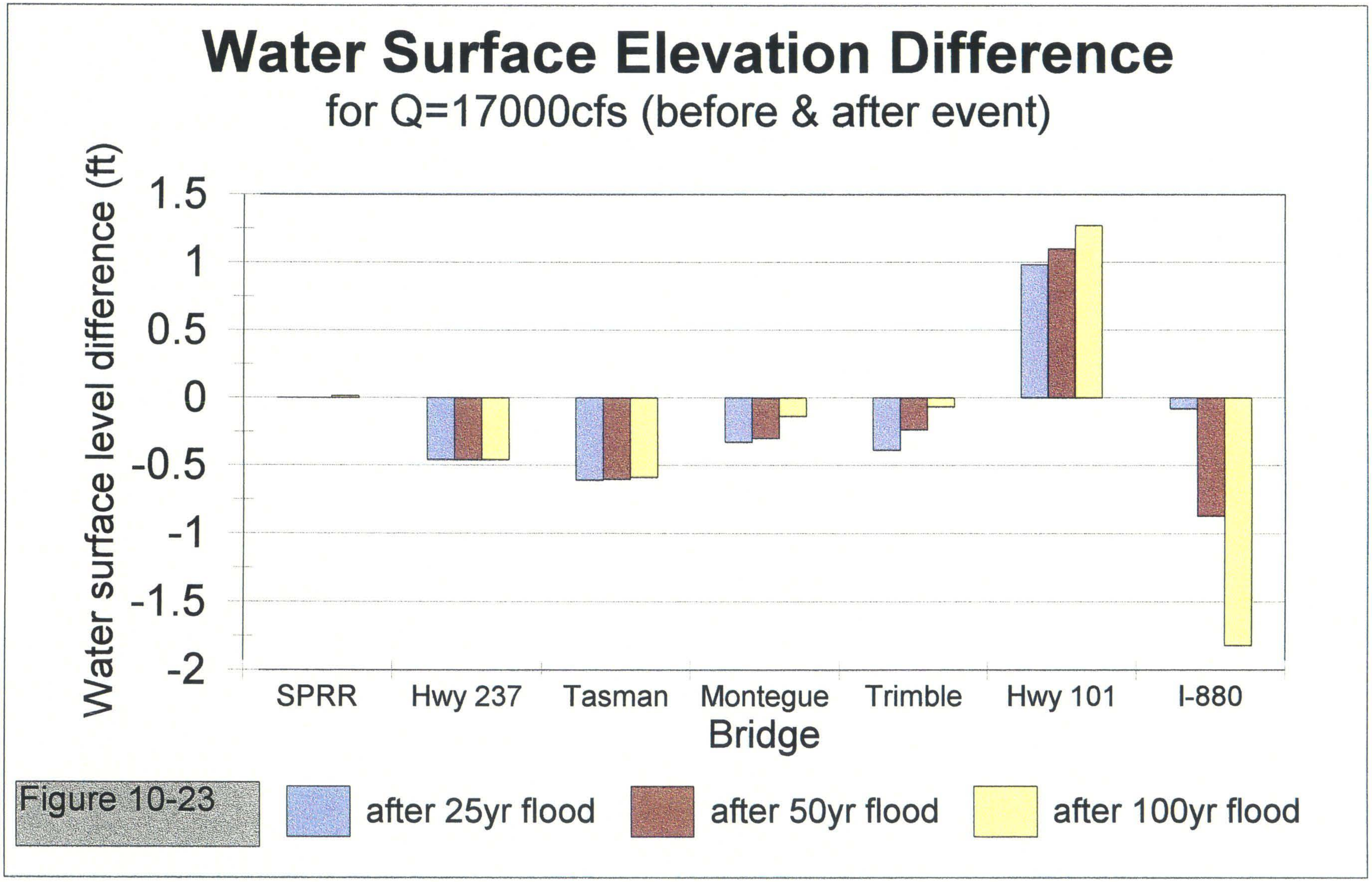


Figure 10-23

Water Surface Elevation Difference
for Q=17000cfs (before & after event)

Sedimentation Analysis of COE and SCVWD Design Hydrographs

11. Sedimentation Analysis of the Corps of Engineers and Santa Clara Valley Water District Design Hydrographs

The sedimentation modeling analysis presented in Chapters 9 and 10 documents the most detailed sedimentation modeling study completed to date for the lower Guadalupe River system. The analysis includes an assessment of long-term changes in (i.e. multi-decade) sediment dynamics and stage-discharge relationships on the lower Guadalupe River, as well as an assessment of individual T-year recurrence interval floods on sediment dynamics and stage-discharge relationships. As shown in the sedimentation modeling analysis reported in Chapters 9 and 10 and the long-term aggradation data developed from the cross section comparisons provided in Appendix A, long term sedimentation modeling simulations are key for quantifying the magnitude and locations of sediment aggradation within the project reach. The long-term simulations also provide a means of quantifying the increase in flood risk due to reduction in channel capacity of the study reach. In addition, long-term sedimentation modeling provides a means for quantifying sediment deposition for use in developing a channel maintenance program for maintaining flood conveyance capacity in the Lower Guadalupe River. Individual recurrence interval flood hydrographs were developed in Chapter 9 for quantifying the sedimentation response of the lower Guadalupe River to individual flood events. The shape and duration of these flood hydrographs were developed specifically for analyzing sedimentation processes, and were developed from characteristics of observed flood events on the Lower Guadalupe River. The hydrographs presented in Chapter 9 depict an integration of actual flood events representative of existing watershed conditions.

For flood control design purposes, synthetic storm flood hydrographs for the 100-year recurrence interval flood have been developed by the US Army Corps of Engineers (Corps of Engineers, 1977) and the Santa Clara Valley Water District (SCVWD, 1997). These flood hydrographs were developed using techniques different from that described in Chapter 9, and were developed specifically for analysis and design of flood control project alternatives. A comparison of the 100-year 8-day, average daily flood hydrograph developed for the sedimentation analysis portion of this study, the Corps of Engineers 100-year flood hydrograph, and the Santa Clara Valley Water District 100-year hydrograph is graphically illustrated on Figure 11-1. The hydrographs developed by the Corps of Engineers and the SCVWD indicate higher peak flowrates than the 8-day daily average flood hydrograph, and a much shorter event duration than the 8-day daily average hydrograph developed from observed flood hydrographs on the lower Guadalupe River system.

Sedimentation dynamics using the Corps of Engineers and SCVWD 100-year flood hydrographs was computed with the HEC-6T model for existing conditions, and for the project Alternatives 2 and 3 described in Chapter 10. Results from these simulations were compared with the results from the 8-day daily average 100-year flood for existing conditions and Alternatives 2 and 3. The total sediment delivery of these individual events for each condition is summarized in Table 11-1.

Table 11-1. Comparison of Accumulated Sediment Loads - Corps of Engineers, Santa Clara Valley Water District, and 8-day duration 100-year Flood Hydrographs

		Existing Conditions					
		Corps of Engineers flood hydrograph		SCVWD flood hydrograph		8-day Duration flood hydrograph	
Bridge	Location	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)
SPRR	22970	2655	146788	2687	131156	3694	369517
Hyw 237	26343	3503	146788	3515	131156	5077	369517
Tasman	30255	4714	146788	4761	131156	7109	369517
Montegue	38736	17175	146788	16908	131156	26114	369517
Trimble	44904	45585	146782	42103	131148	76750	369510
Hyw 101	47578	54319	146771	50350	131136	86527	369490
I-880	57896	30693	146692	28779	131069	73634	369392
		Alternative 2					
		Corps of Engineers flood hydrograph		SCVWD flood hydrograph		8-day Duration flood hydrograph	
Bridge	Location	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)
SPRR	22970	2548	146787	2575	131147	3498	369517
Hyw 237	26343	3374	146787	3383	131147	4836	369517
Tasman	30255	4571	146787	4596	131147	6838	369517
Montegue	38736	16339	146787	15909	131147	25602	369508
Trimble	44904	45925	146782	42988	131143	76337	369500
Hyw 101	47578	53967	146766	50315	131126	85052	369481
I-880	57896	30830	146692	28097	131061	73367	369392
		Alternative 3					
		Corps of Engineers flood hydrograph		SCVWD flood hydrograph		8-day Duration flood hydrograph	
Bridge	Location	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)	sand (tons)	silt&clay (tons)
SPRR	22970	2467	146787	2391	131136	3365	369519
Hyw 237	26343	3279	146787	3157	131136	4666	369519
Tasman	30255	4471	146787	4309	131136	6638	369519
Montegue	38736	16410	146787	15528	131136	25545	369519
Trimble	44904	47445	146784	43955	131132	79038	369514
Hyw 101	47578	54482	146768	49933	131120	86503	369492
I-880	57896	30901	146692	27291	131059	73692	369391

As shown in Table 11-1, the total sediment load passing key locations in the lower Guadalupe River show similar trends for the three alternate flood hydrographs and three project alternatives analyzed. First, fine sediment load (silt and clay) pass through the study reach as washload, and little deposition of these materials occurs during this large magnitude flood event for any project alternative in the Guadalupe River upstream of the SPRR bridge. Second, project alternatives 2 and 3 described in Chapter 10 have similar effects on computed sediment loads for the three 100-year flood hydrographs analyzed. Alternatives 2 and 3 result in small increases and downstream redistribution of sediment storage in the Guadalupe River between the SPRR and I-880 bridges for the three flood hydrographs.

The effects of flood hydrograph duration on total event sediment load are clearly evident in Table 11-1. The Corps of Engineers and the Santa Clara Valley Water District 100-year flood hydrographs shown in Figure 11-1 are 2 day hydrographs, in contrast to the 8-day duration hydrographs developed from analysis of significant runoff events that occurred on the lower Guadalupe River in the last two decades. The shorter duration of the Corps of Engineers and Santa Clara Valley Water District flood hydrographs result in lower total sediment loads (and resultant sediment deposition) than that computed for the 8-day duration flood hydrograph. The sedimentation response of the lower Guadalupe River will vary depending on the magnitude and duration of the hydrologic event imposed on the system. Hydraulic design of flood control alternatives can be based on the shorter duration synthetic flood hydrographs developed by the Corps of Engineers or Santa Clara Valley Water District, however, NHC recommends that project performance and sediment maintenance requirements be based on long term sedimentation simulations. As discussed in Chapters 8, 9, and 10, long-term (i.e. multi decade) simulation of the sedimentation behavior of the lower Guadalupe River provides a quantifiable verification check on the sedimentation model performance (Figure 8-4). In addition, long-term simulation also provides quantitative changes in stage-discharge rating curves, and hence flood risk, for the lower Guadalupe River.

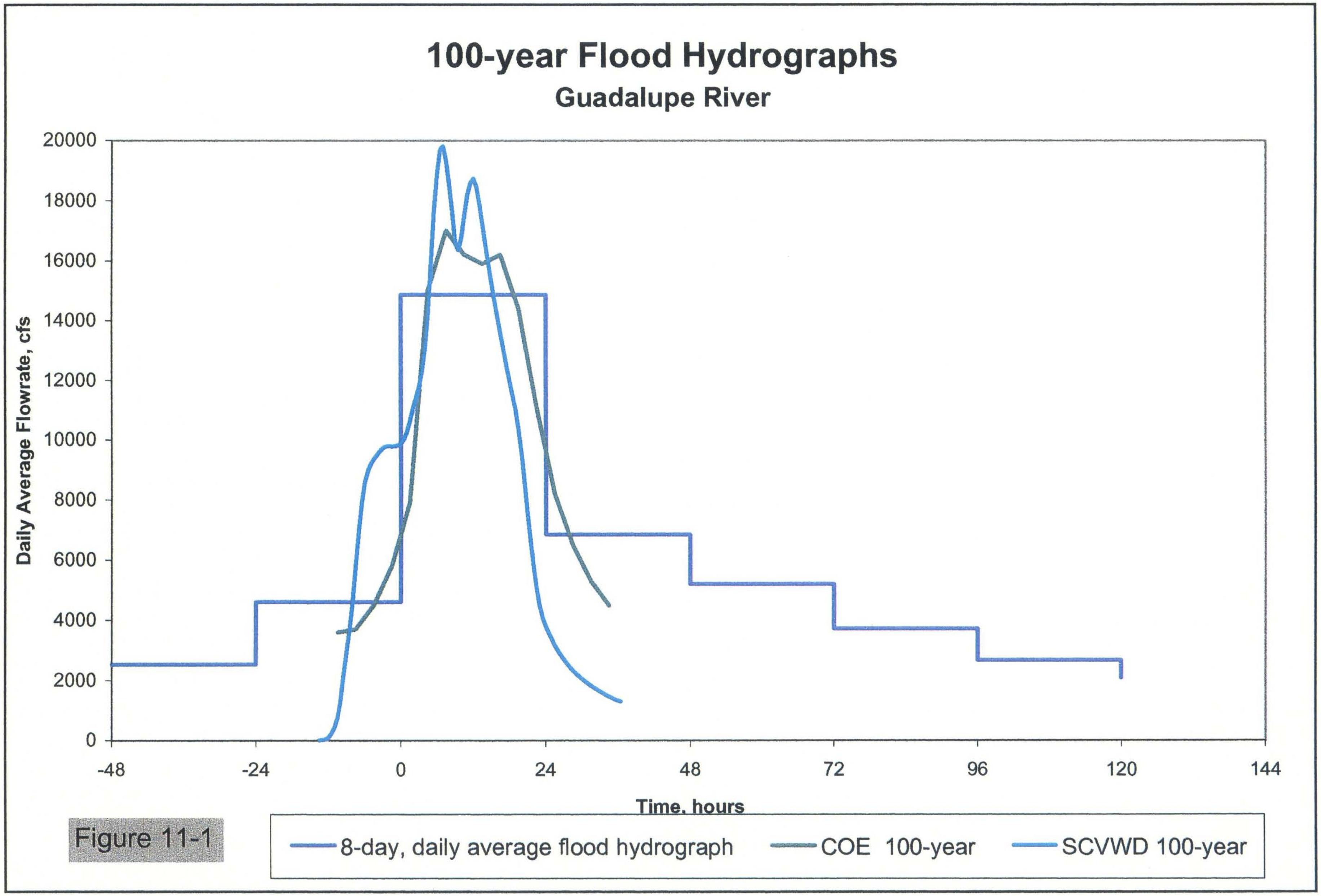

100-year Flood Hydrographs
Guadalupe River
Daily Average Flowrate, cfs
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
-48
-24
0
24
48
72
96
120
144
Time, hours
8-day, daily average flood hydrograph
COE 100-year
SCVWD 100-year

Figure 11-1

Chapter 12

Conclusions

12. Conclusions

Following are conclusions developed from the results of this investigation of the Lower Guadalupe River:

- The present Lower Guadalupe River cannot convey the 100-year flood event.
- Data show significant reduction in channel capacity since construction of the flood control project.
- The four primary contributors to the reduction in channel capacity are;
  1. Watershed derived sediment deposition
  2. Tidally influenced sediment movement and accumulation in the lower reaches
  3. Sensitivity to vegetation and debris within the project levees
  4. Bridge constrictions
- Channel hydraulics and sediment transport characteristics throughout the Project reach are influenced by natural and man-induced agents and physical processes, including geology, seismicity, tidal processes, subsidence, rising sea levels, changes in land use, urbanization, and channel improvement projects.
- The majority of the watershed derived bed material load is comprised of sand sized materials. These materials accumulate predominantly in the reach between Highway 101 and Tasman Drive.
- The majority of the coarse gravels are deposited in the vicinity of Airport Parkway to Trimble Road.
- Watershed derived silts and clays deposit in the tidal reaches during low flow periods. Tidal accretion of Bay muds will continue in the lower reaches.
- If no tidal prism increases are created and subsidence continues to be controlled, long term projections estimate approximately 1.0 to 1.5 feet (0.46 m) of rise in the Guadalupe River marsh-channel cross section configuration within the next 50 years. Long term accumulation of watershed derived flood deposits may add an additional 0.10 to 0.5 feet (0.03 to 0.15 m) to the marsh-channel cross section configurations near Alviso.
- Without regular sediment maintenance, the long term accumulation of watershed derived sediments within the project reach may increase the 100-year flood stage from 5 to 7.5 feet (1.5 to 2.3 m) in the reach from Hwy 101 to Montague.
- Vegetation management does provide for some flood relief. However, vegetation removal primarily in the overbank will increase the rate of sediment accumulation in the reaches where the highest rates of sediment accumulation presently occur and thus may not provide the best long term solution for the reduction of flood risk.
- Off channel sediment basins have only a small affect on reach averaged sediment dynamics. Sediment deposition is primarily controlled by channel slope.
- Channel excavation will provide temporary improvement of flood conveyance capacity. Maintenance of flood capacity will require continuing excavation in perpetuity.

Chapter 13

Recommendations

13. Recommendations

The following recommendations are based on results of this investigation of the Lower Guadalupe River:

- Guadalupe River flood management plans need to recognize that sediment will continue to accumulate and the channel will aggrade in the project reach and that these rates of accumulation will vary with time. Project designs compatible with aggrading channel dynamics should be developed, including programs to manage watershed sediment production and delivery into the Guadalupe River.
- Designated locations within the channel might be designed as regular high maintenance areas where sediment is encouraged to deposit (in sediment traps) and it can be removed on regular basis. Potential constraints include fish passage and riparian habitat issues.
- Project alternatives should consider channel and levee improvements in conjunction with in-channel vegetation and sediment maintenance and removal.
- Methods for reducing bridge-induced hydraulic impacts warrant further investigation.
- Flood flow bypass options should be considered.
- Detailed assessment of the potential for increasing flood risks to reaches downstream of the Project are required.
- Recommend winter high flow and sediment monitoring at select locations for model validation and refinement of project designs.
- An integrated system-wide evaluation of hydraulic and geomorphic processes affecting the whole Guadalupe River Basin should be undertaken to develop rational, feasible flood and habitat management alternatives of the entire Guadalupe River System.

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14. Cited References in Report

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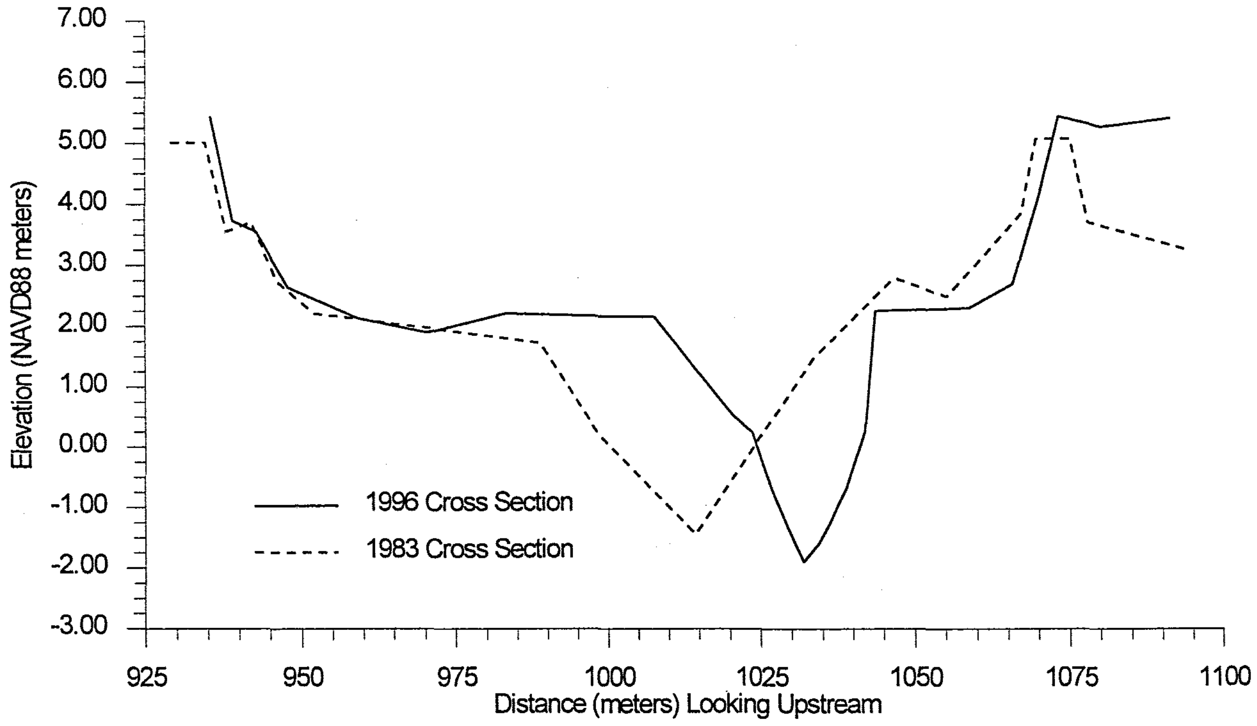
Appendices

APPENDIX A

Comparison of Cross-sections from 1983, 1987 and 1996



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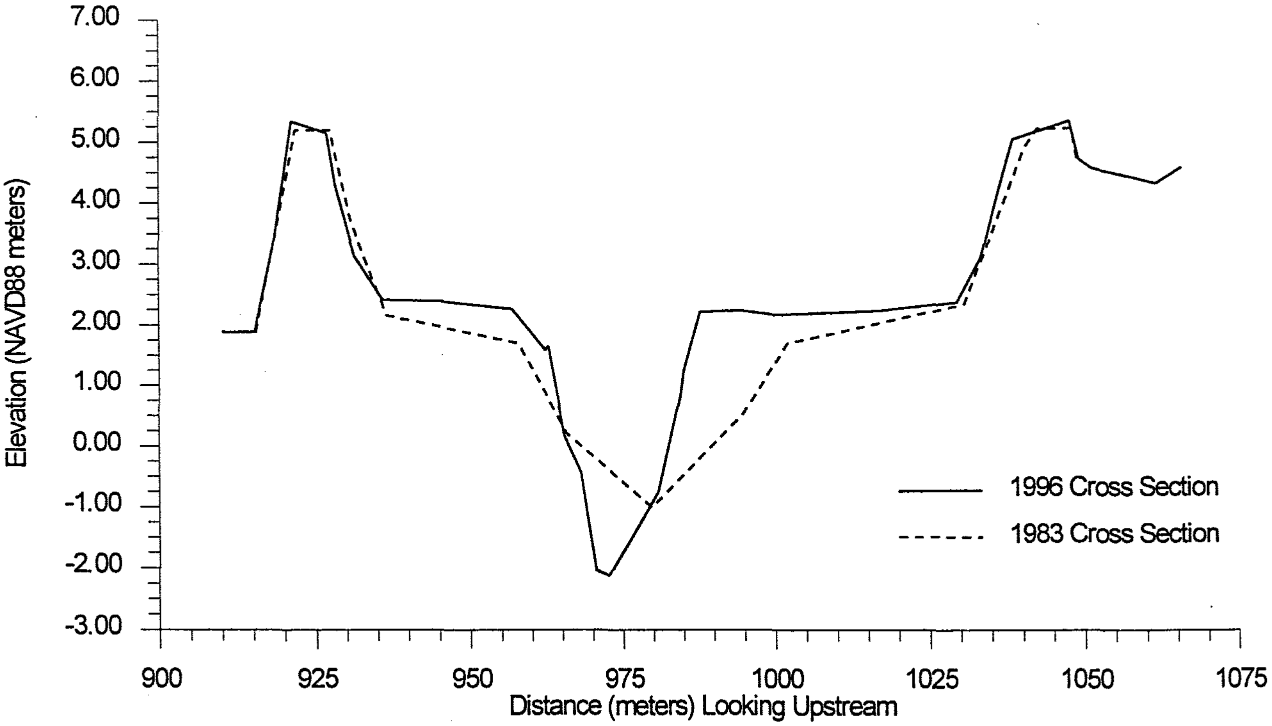

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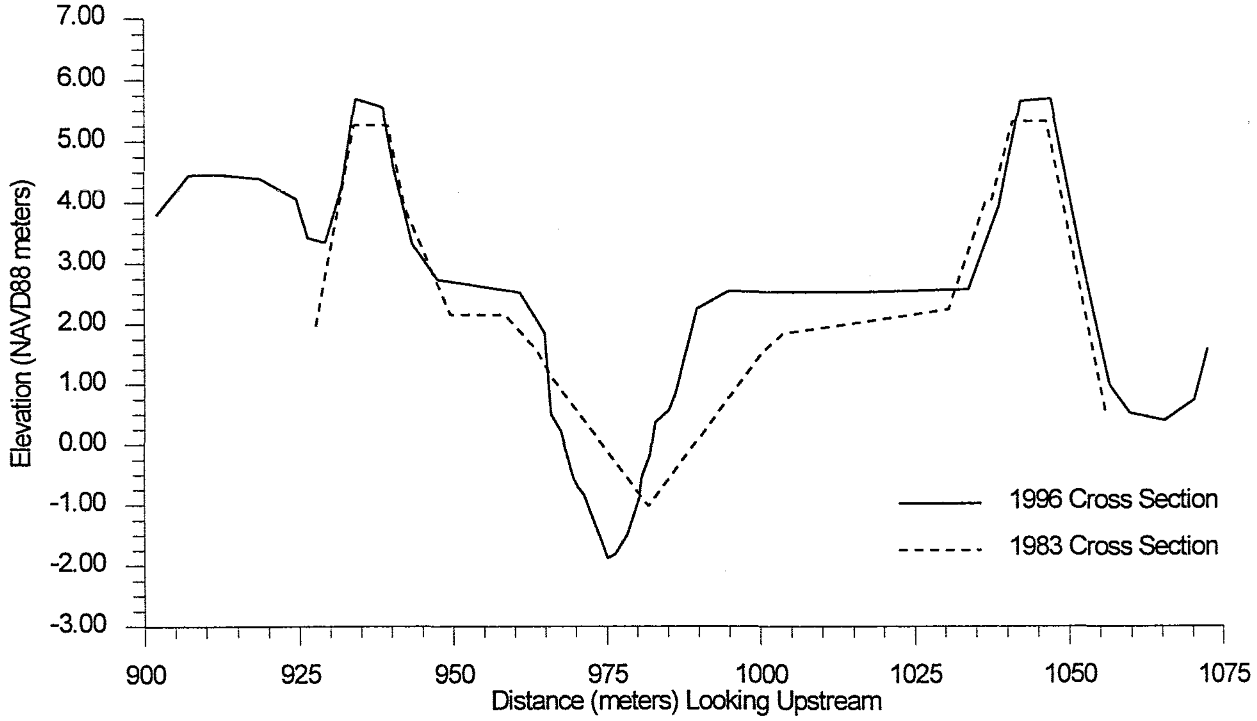

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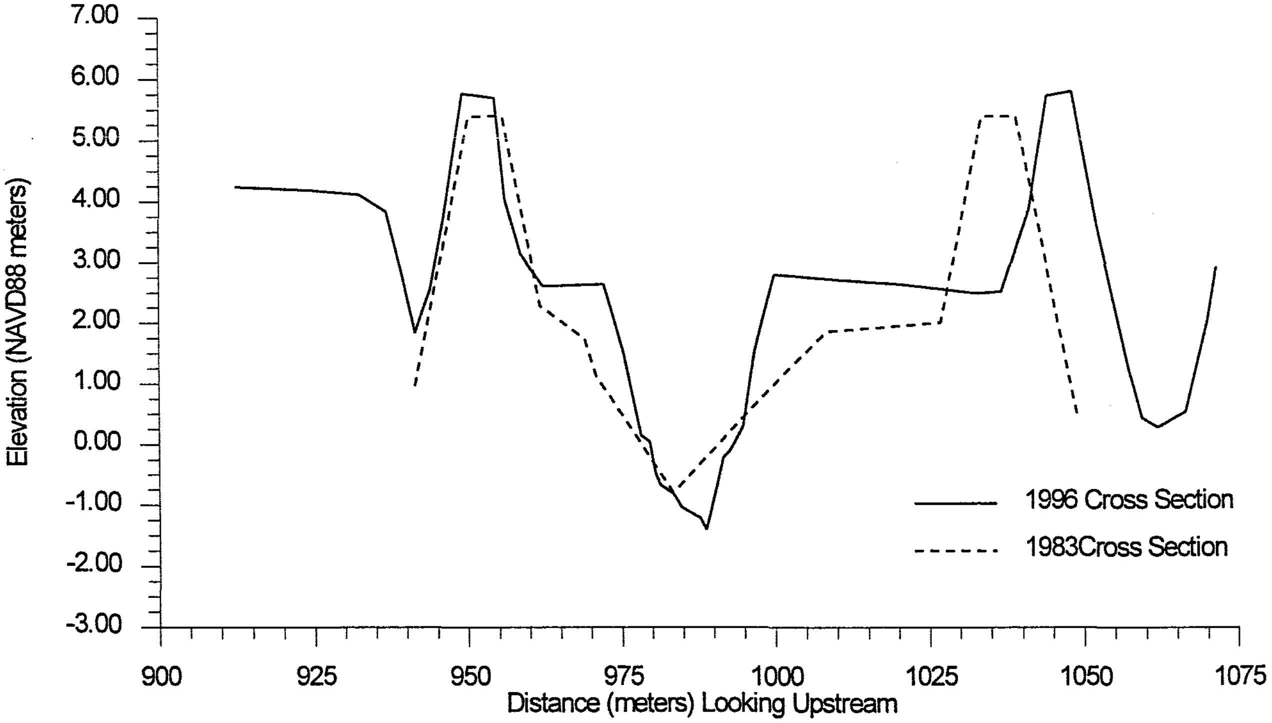

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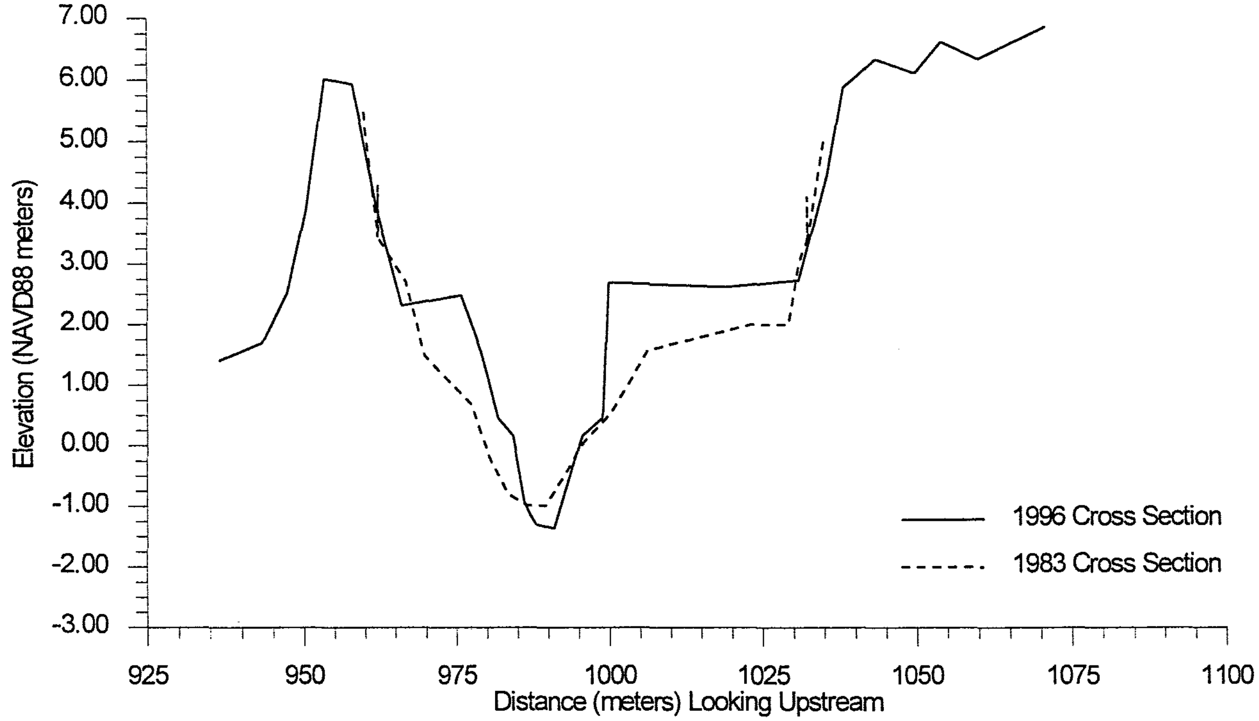

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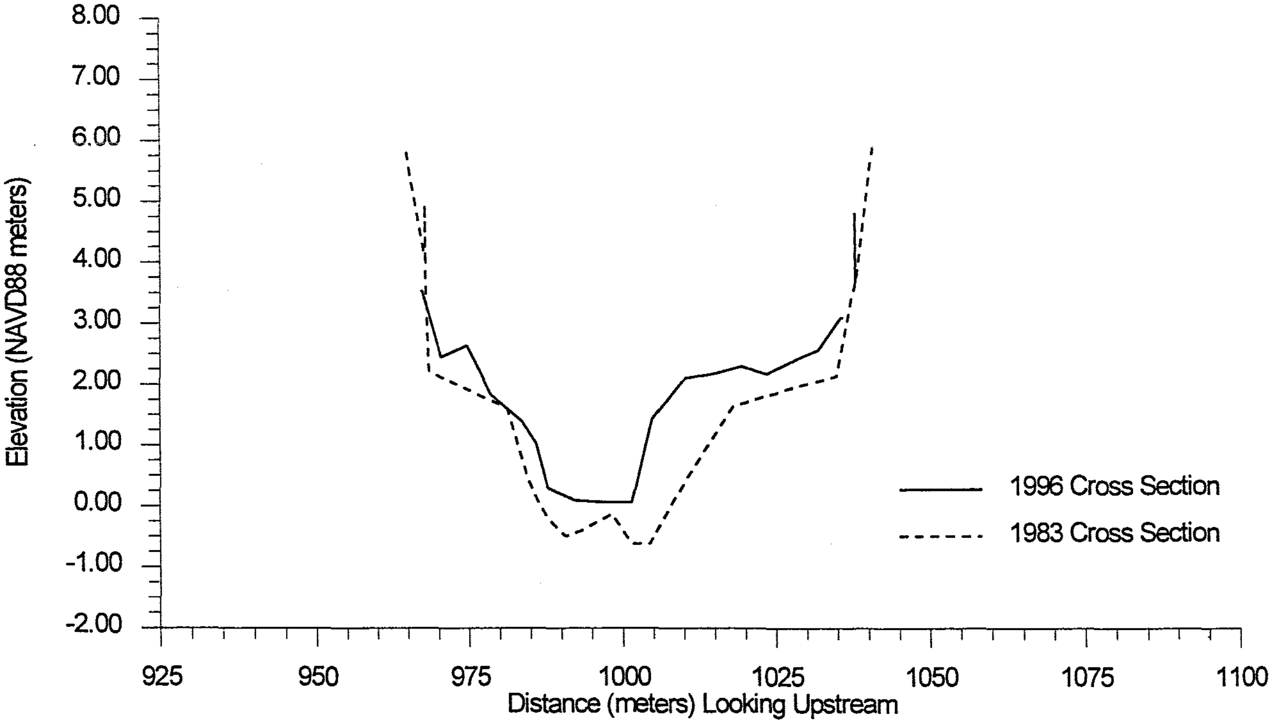

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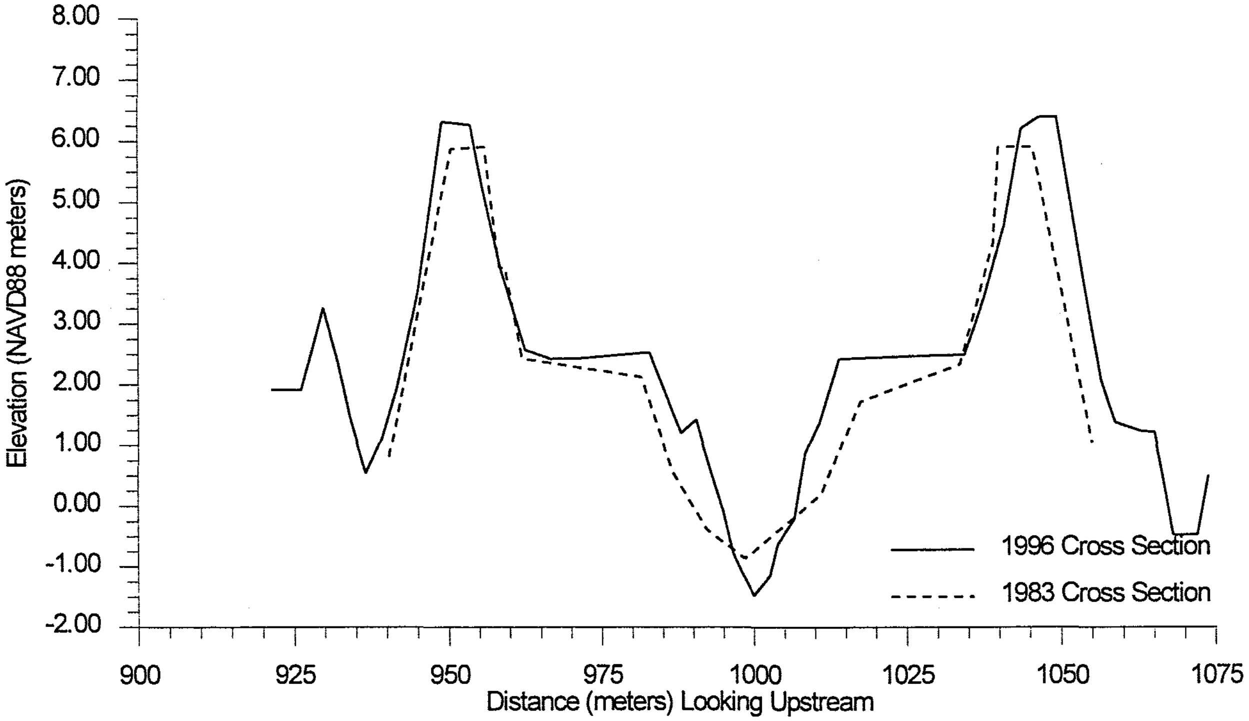

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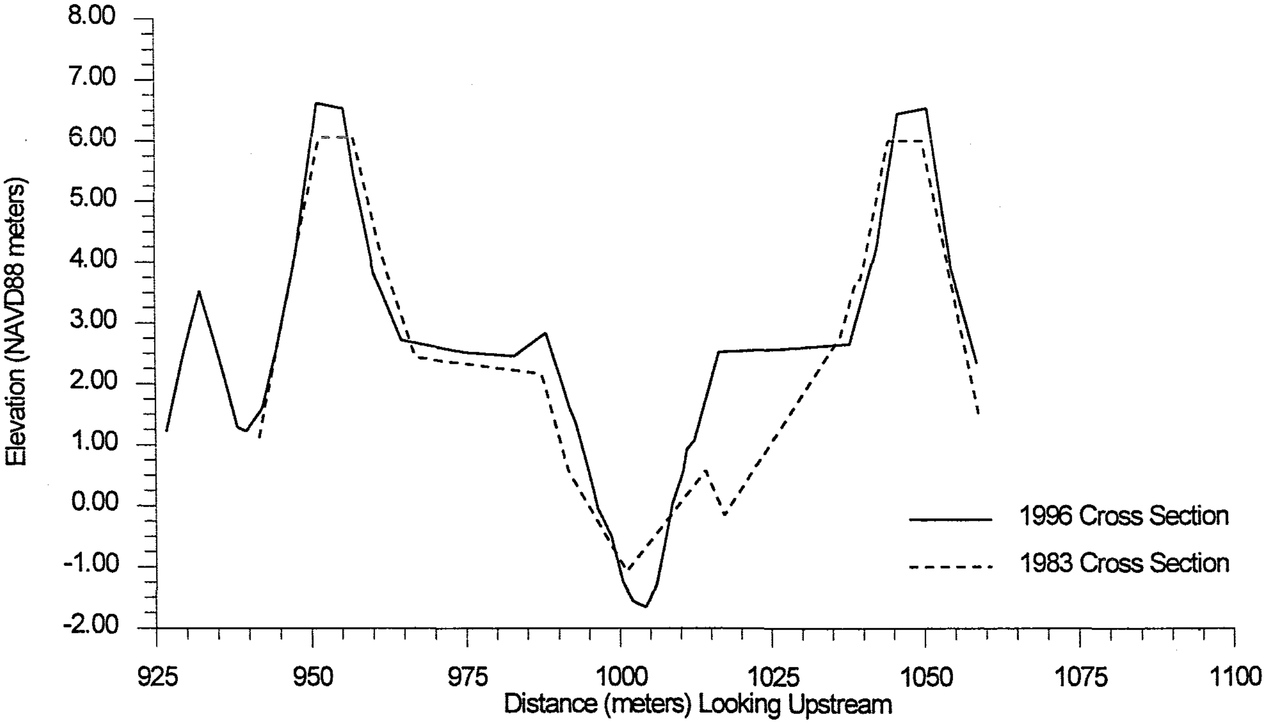

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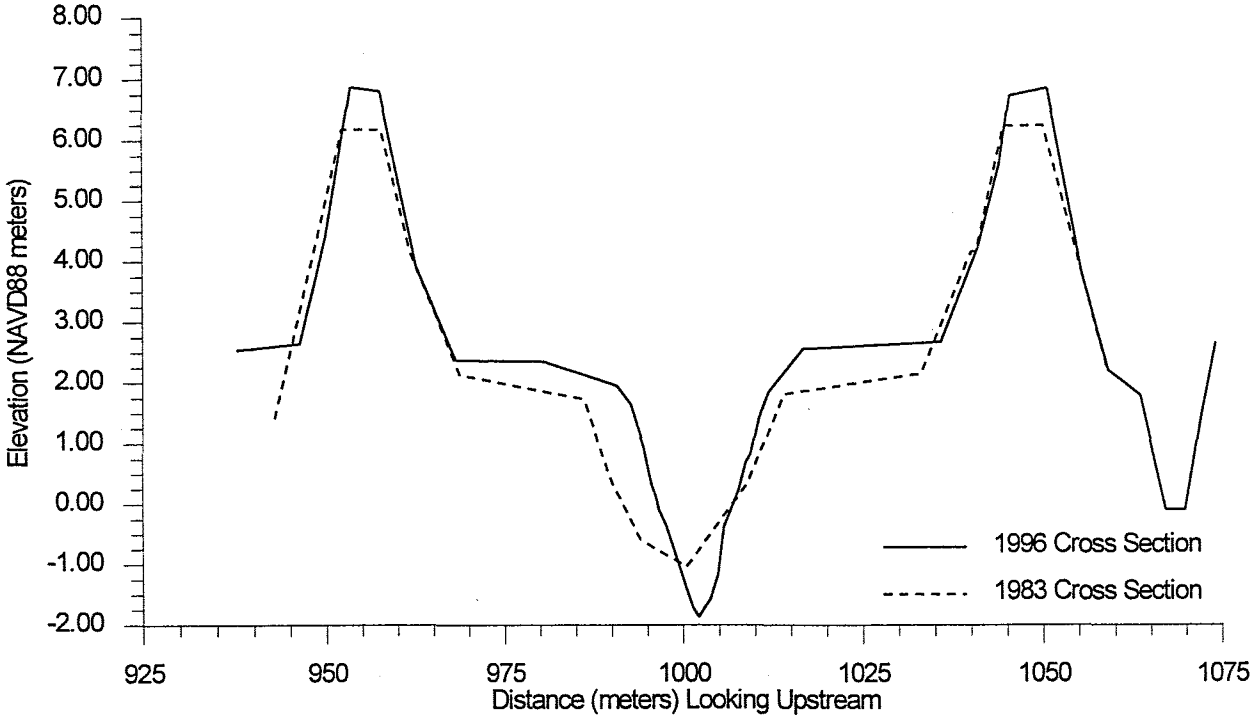

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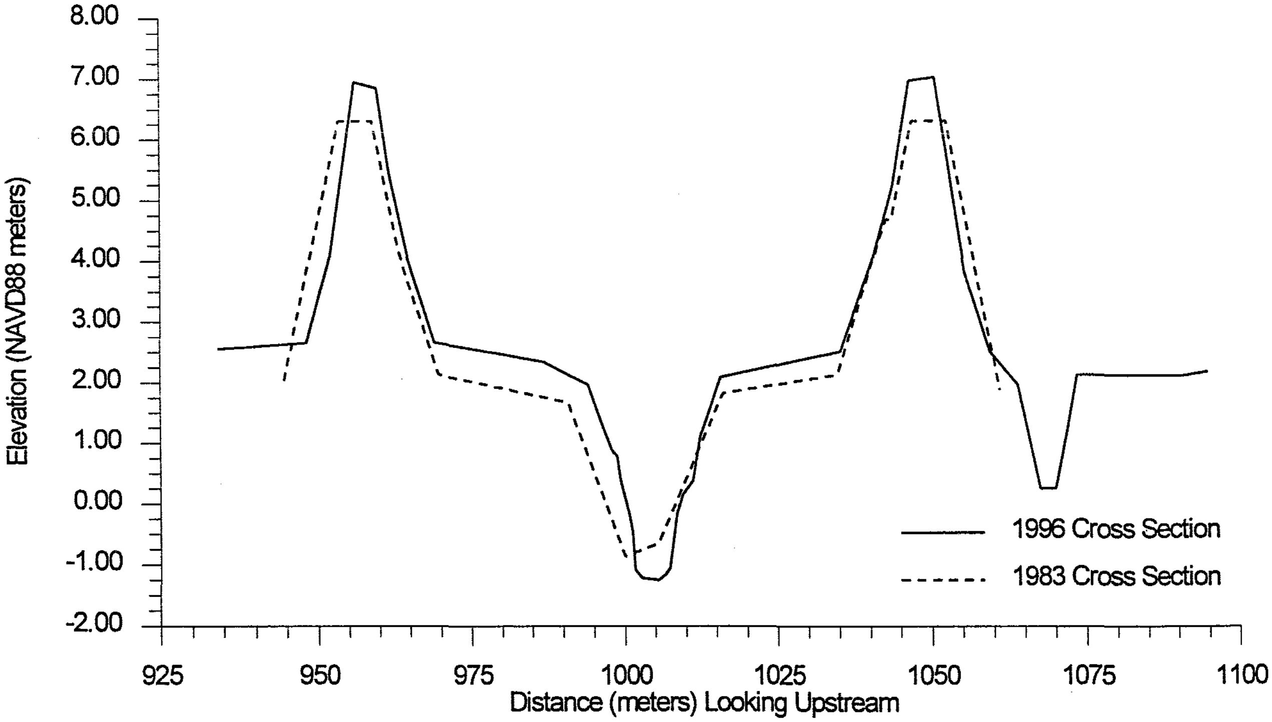

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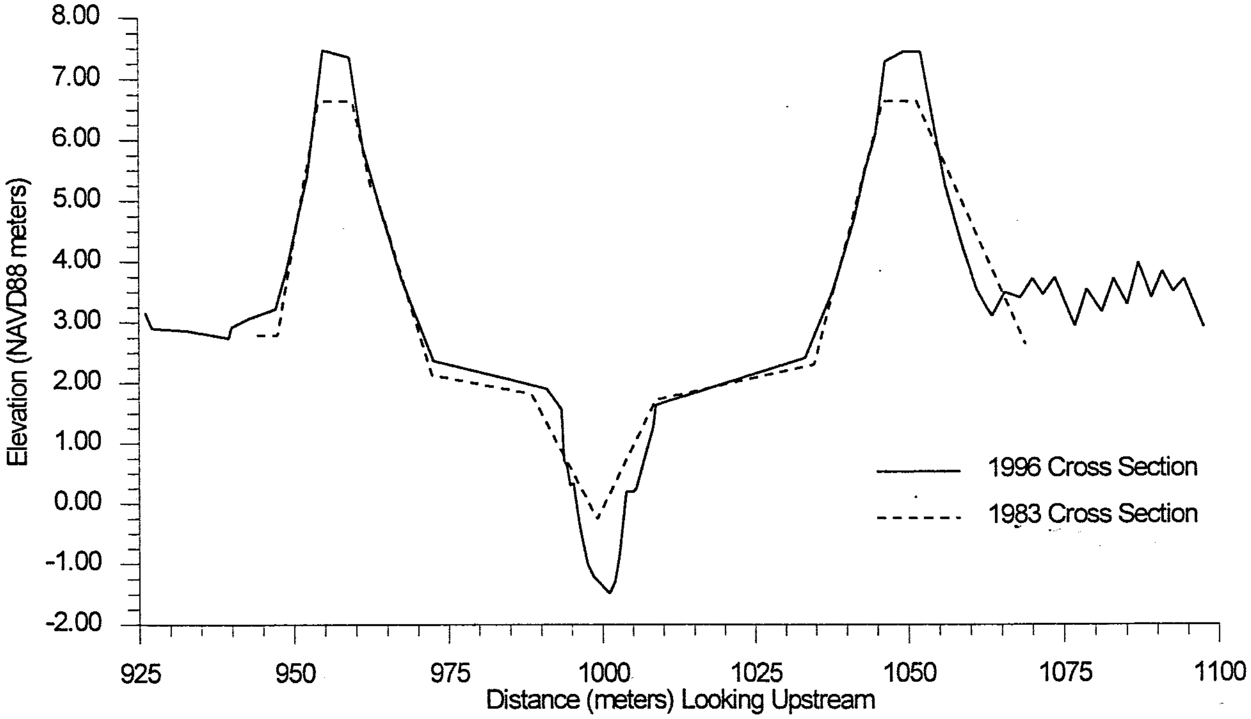

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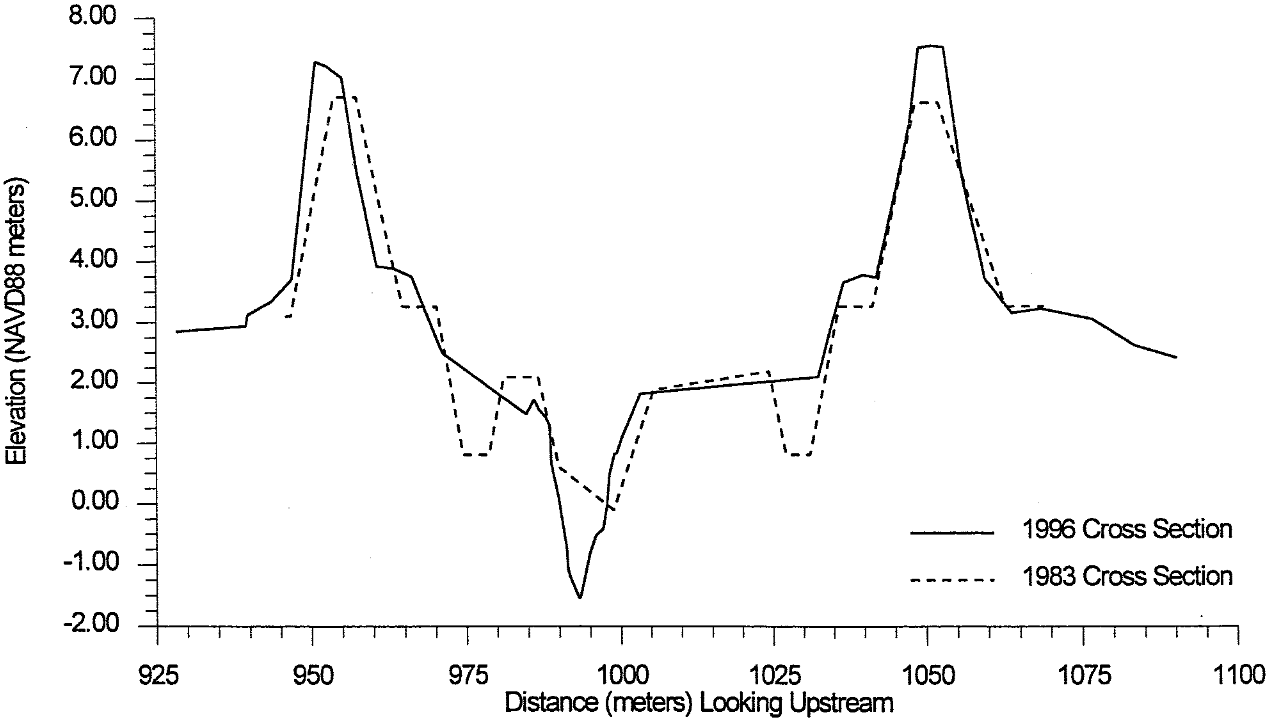

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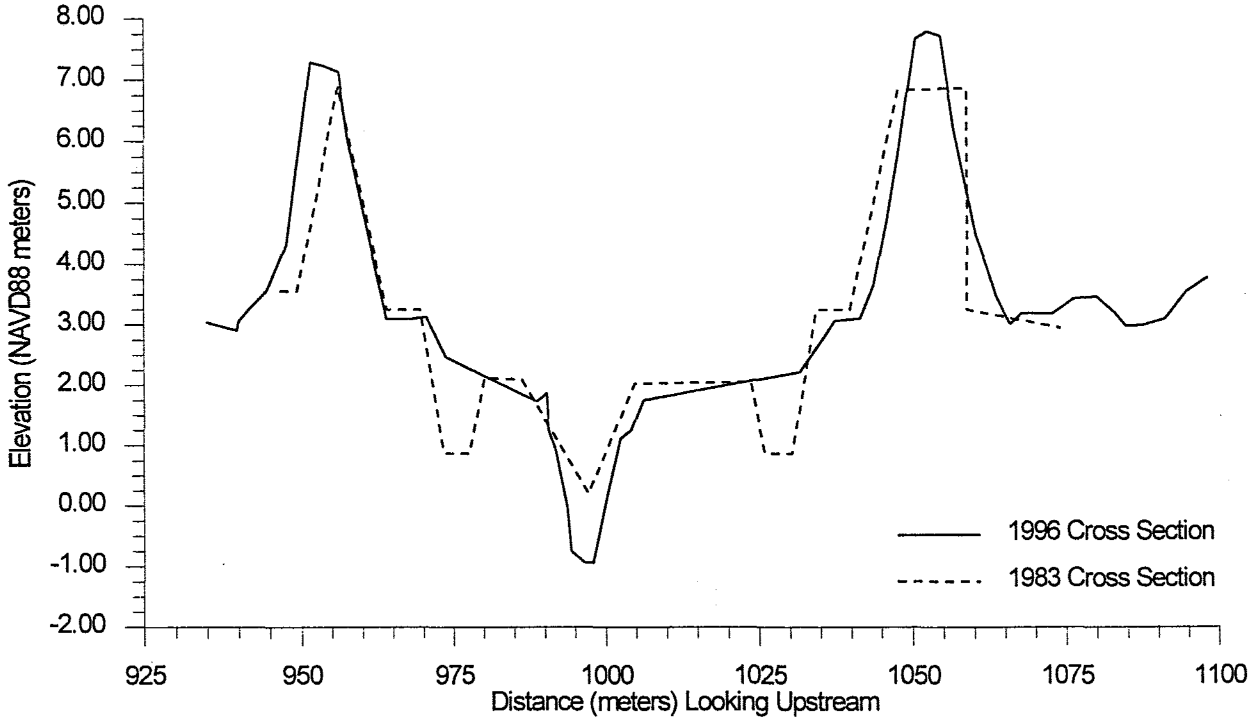

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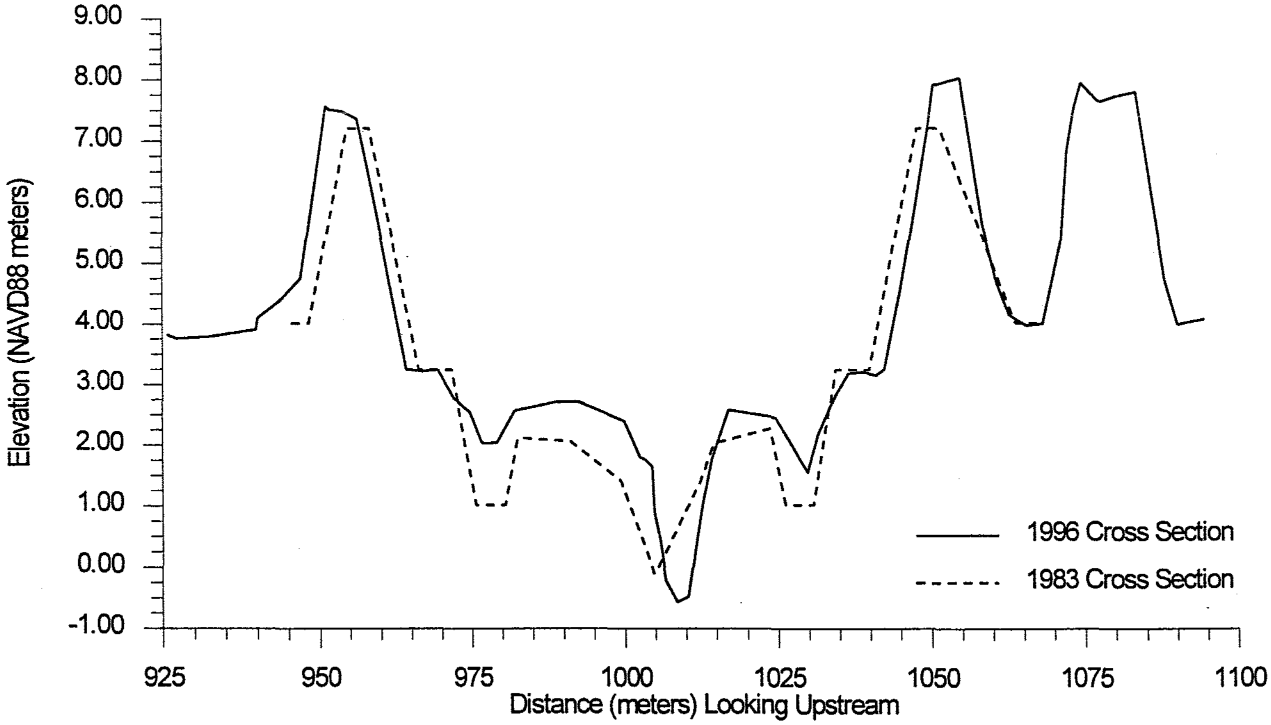

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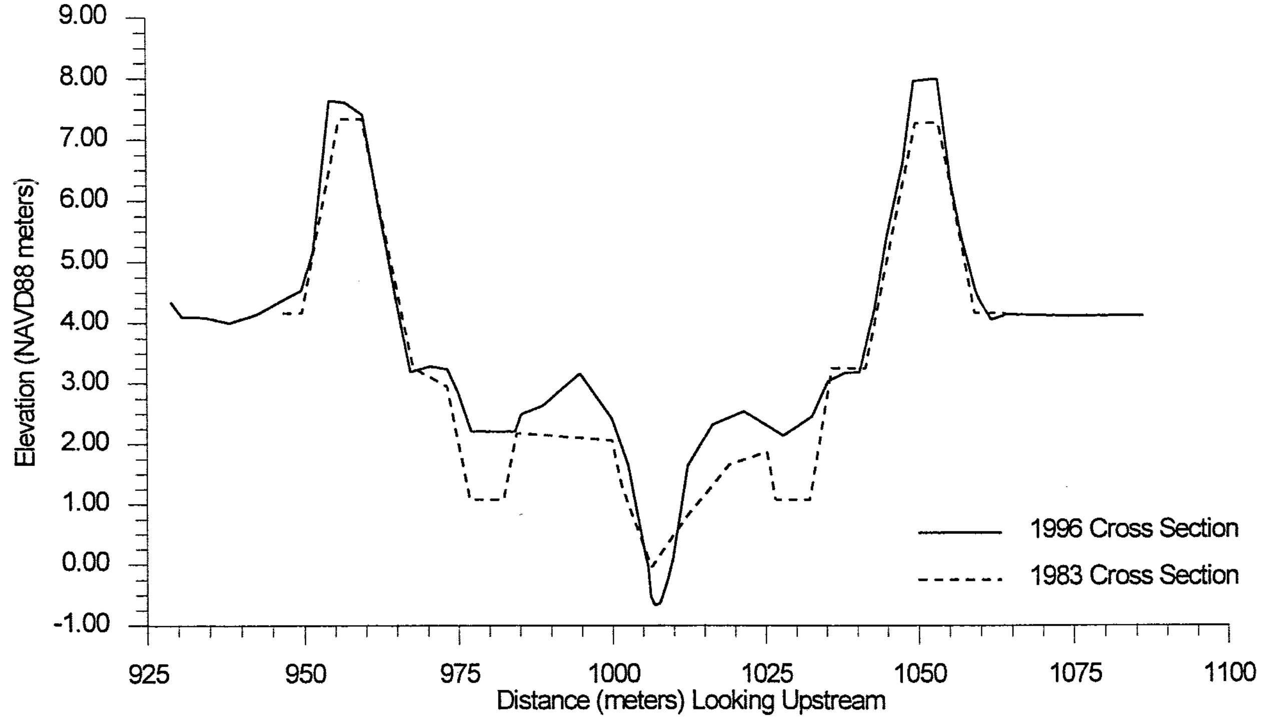

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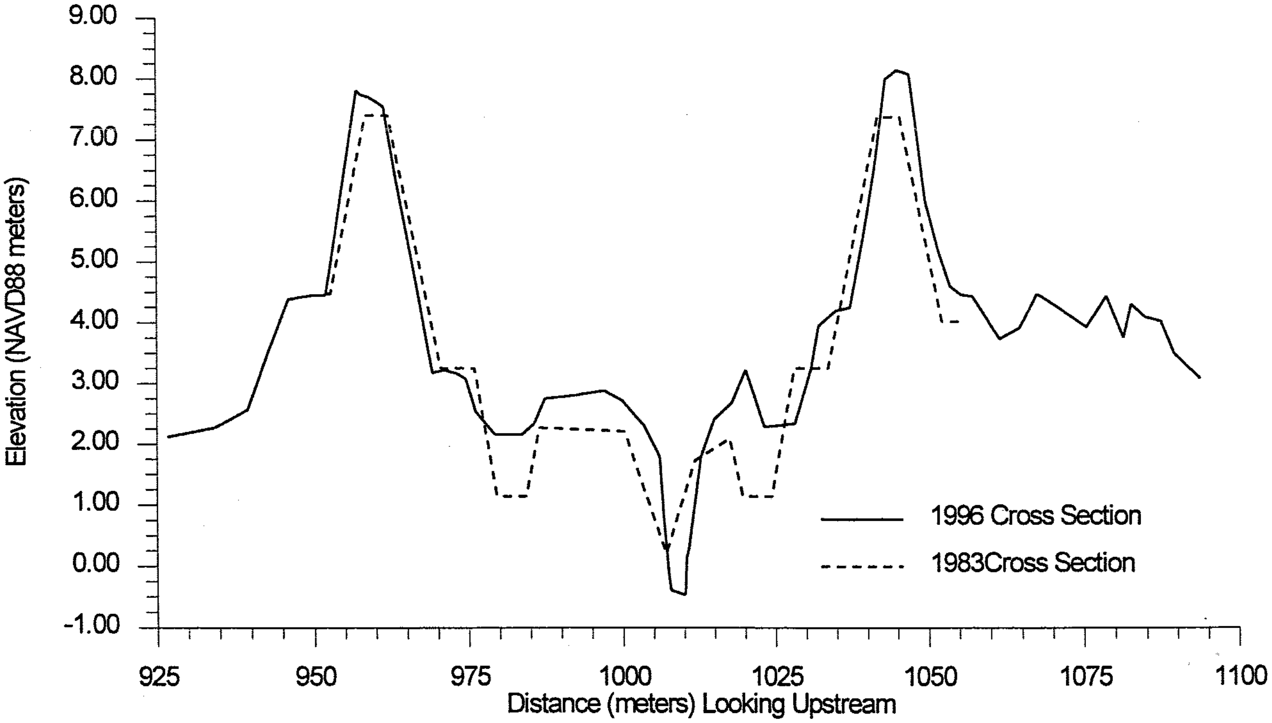

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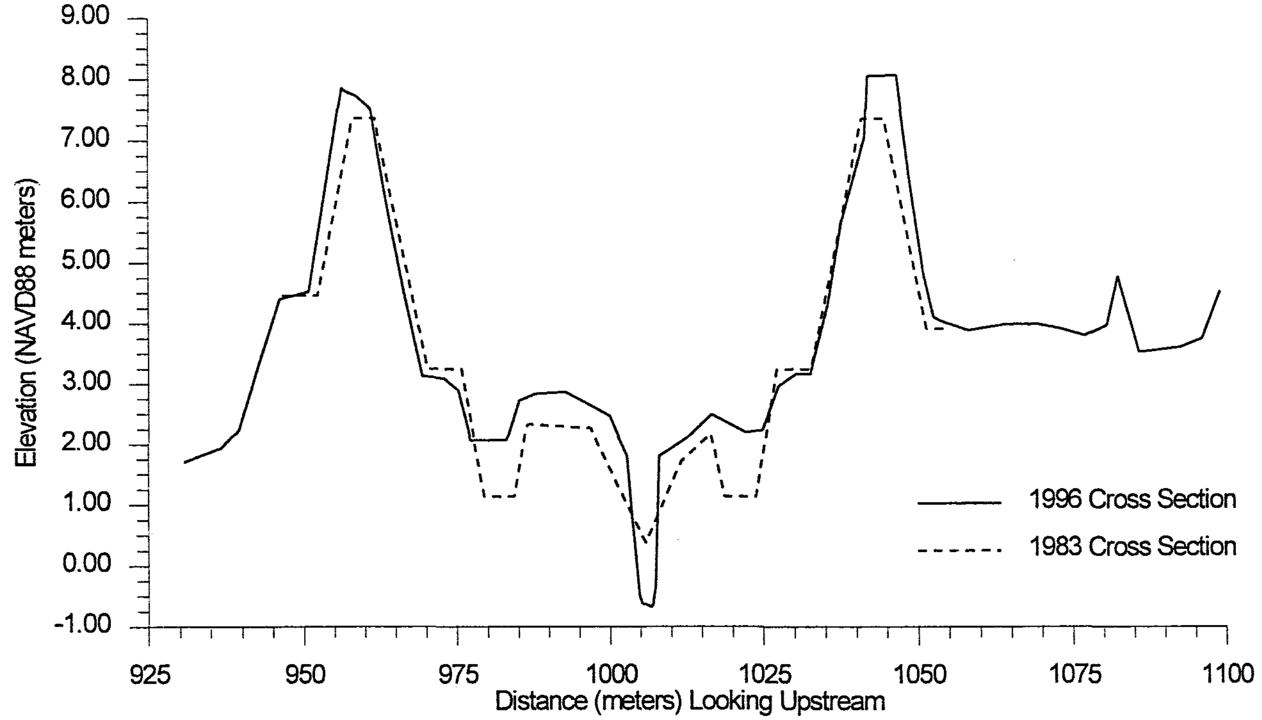

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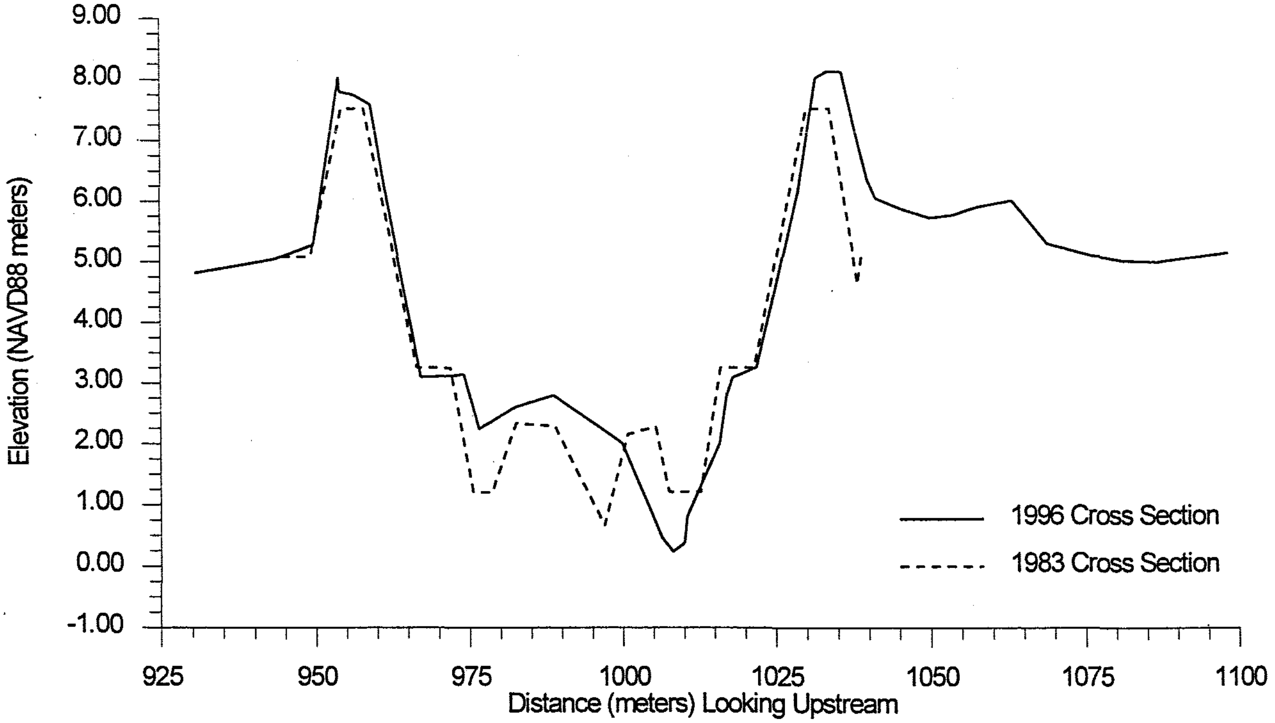

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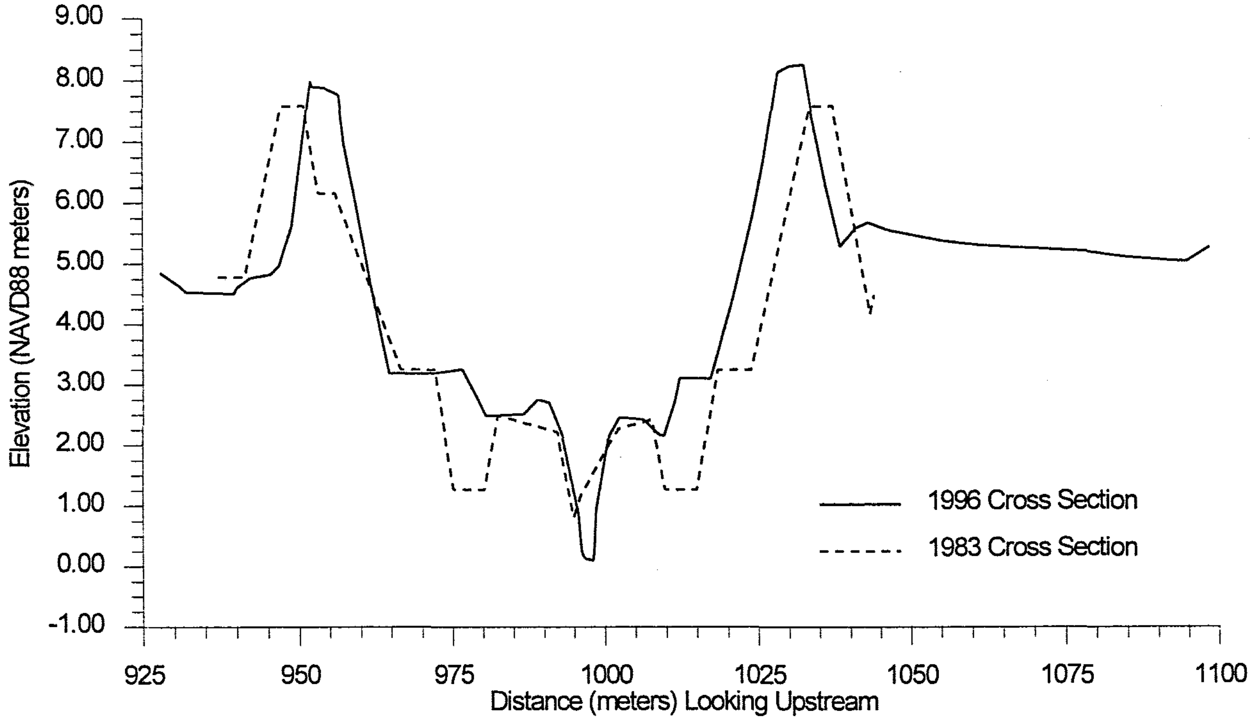

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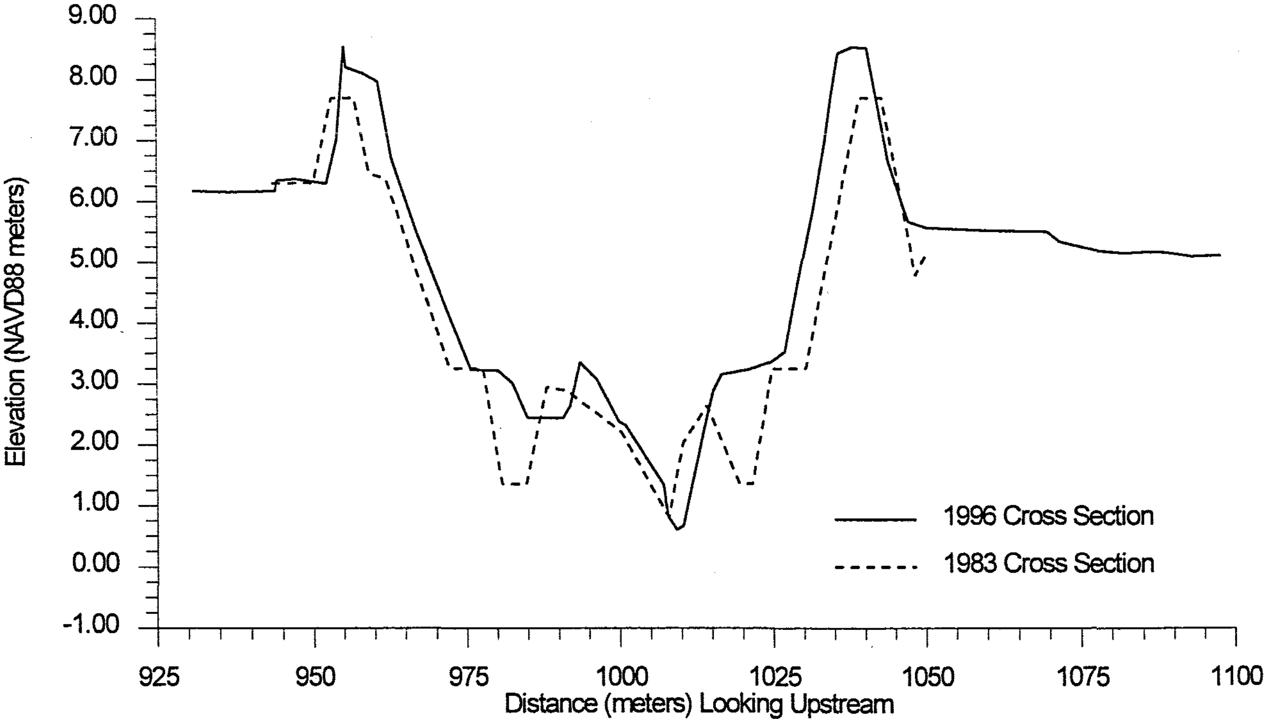

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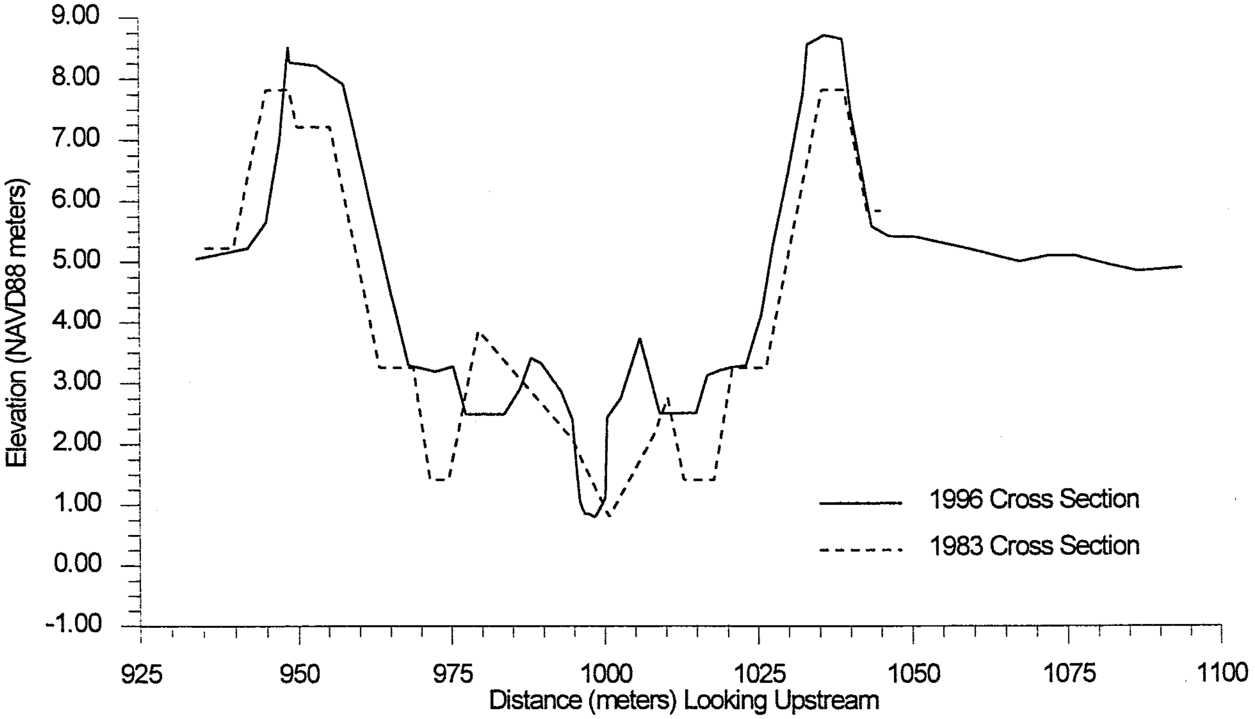

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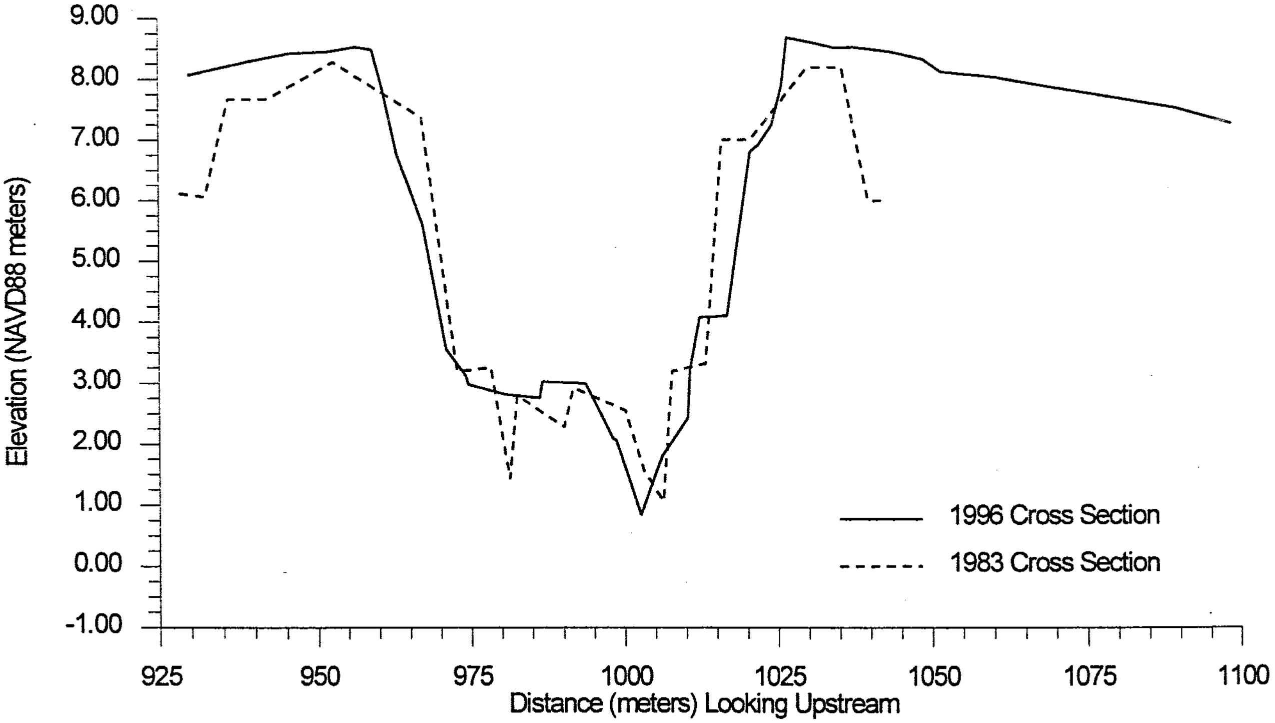

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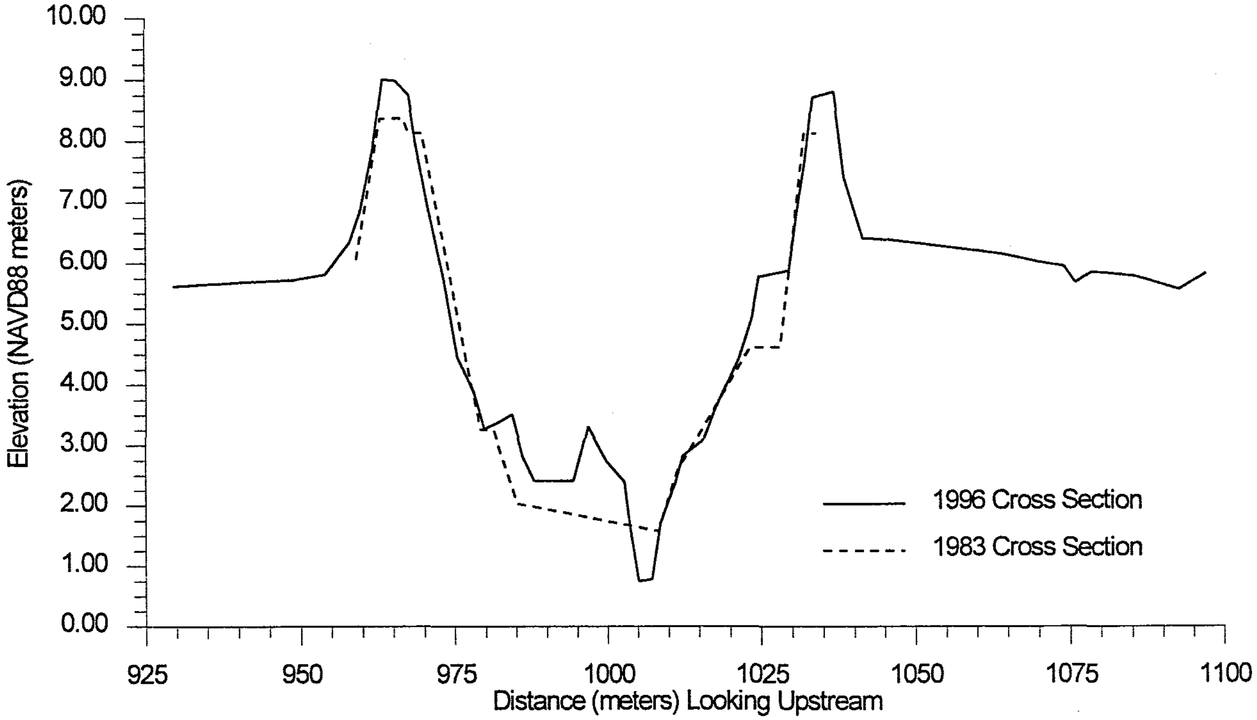

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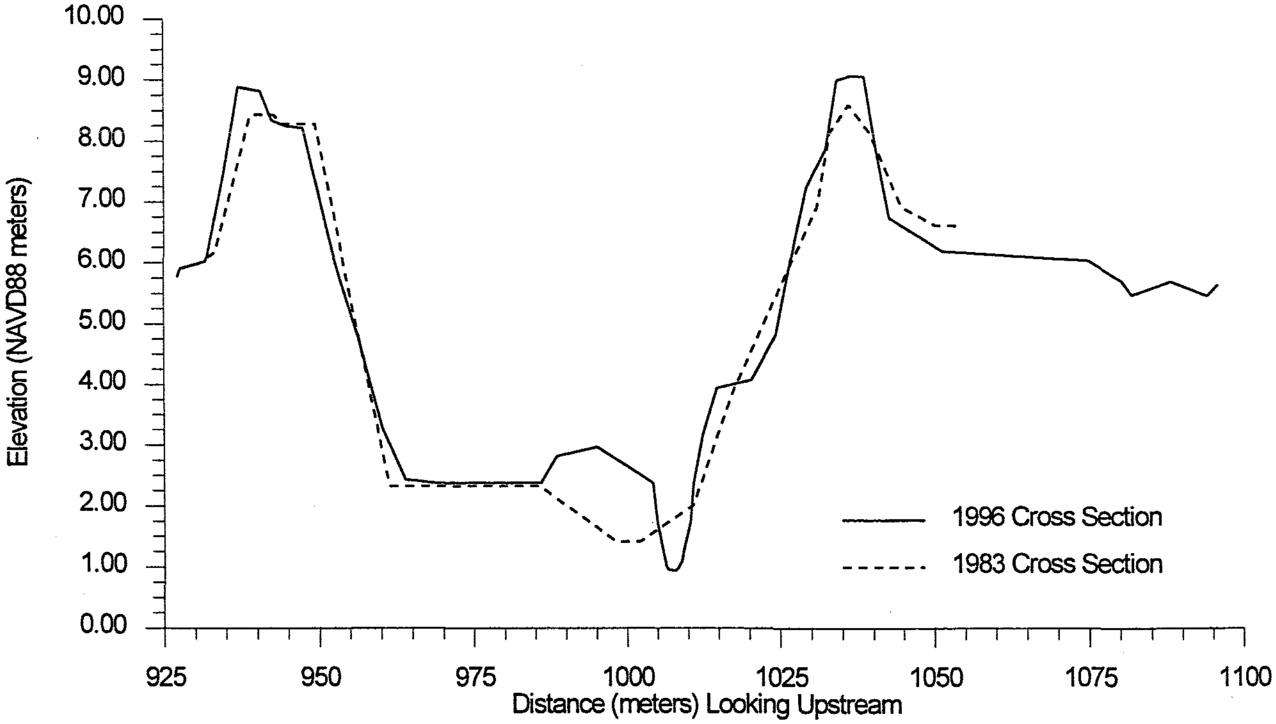

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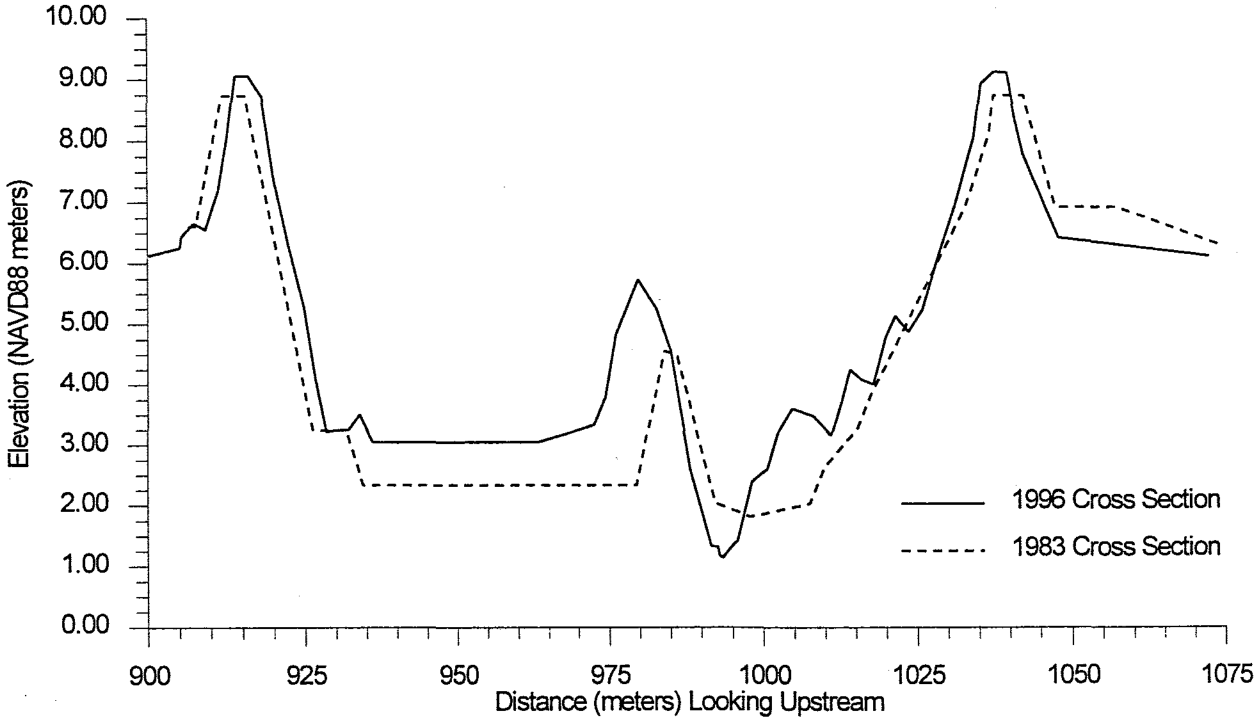

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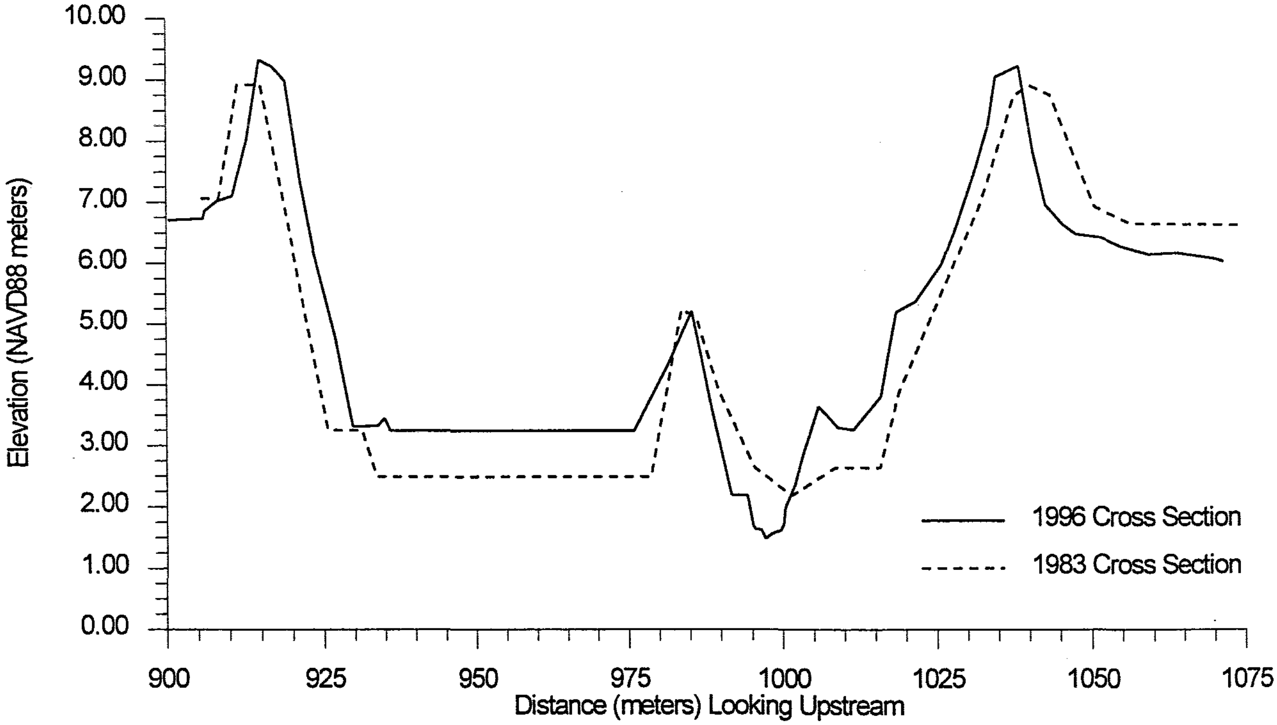

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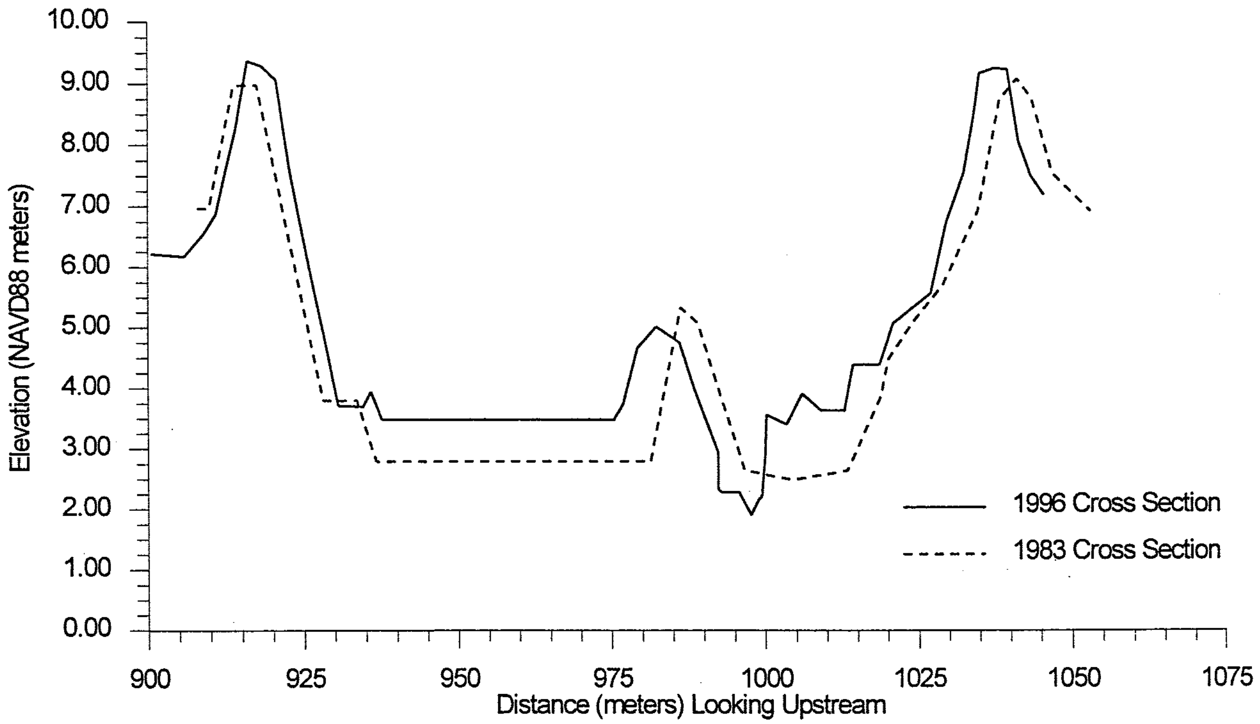

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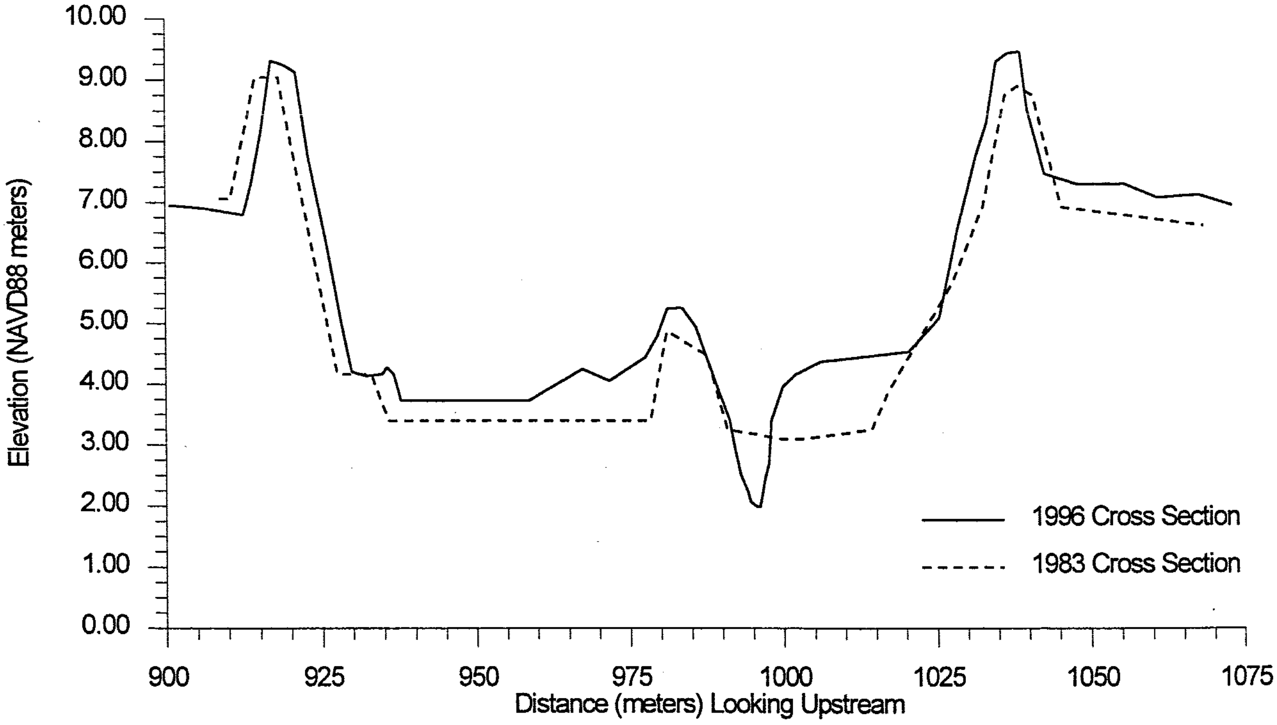

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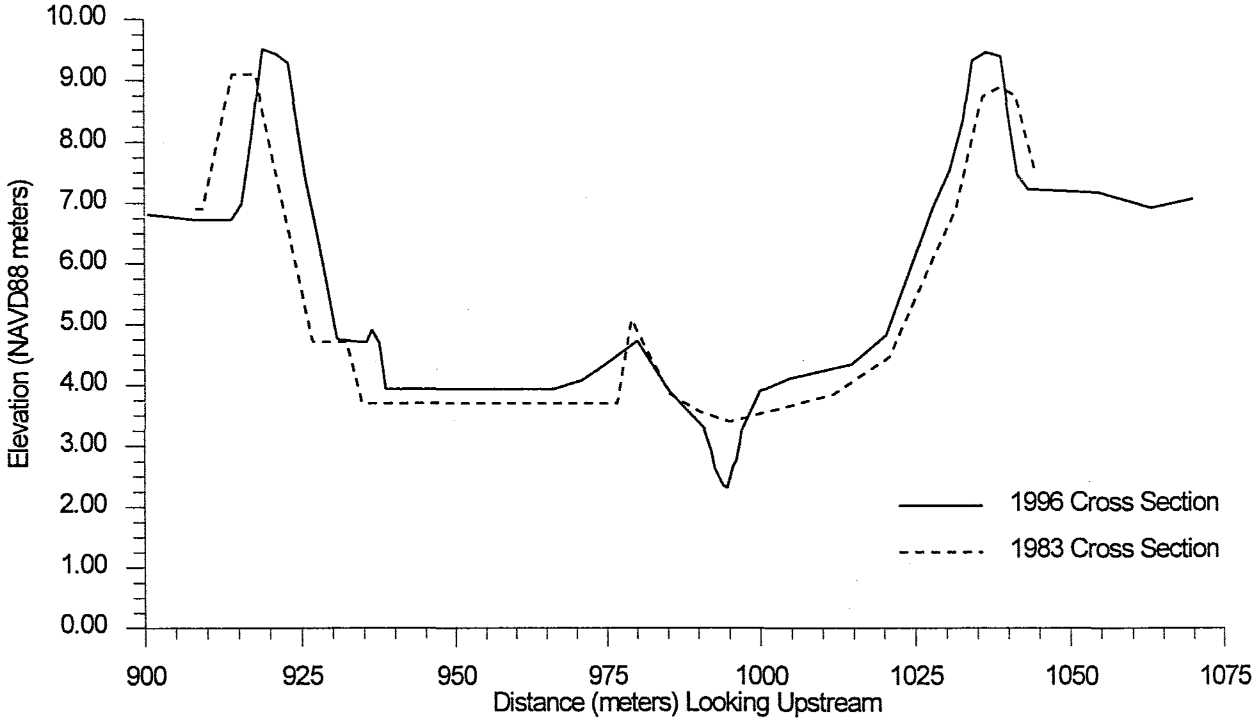

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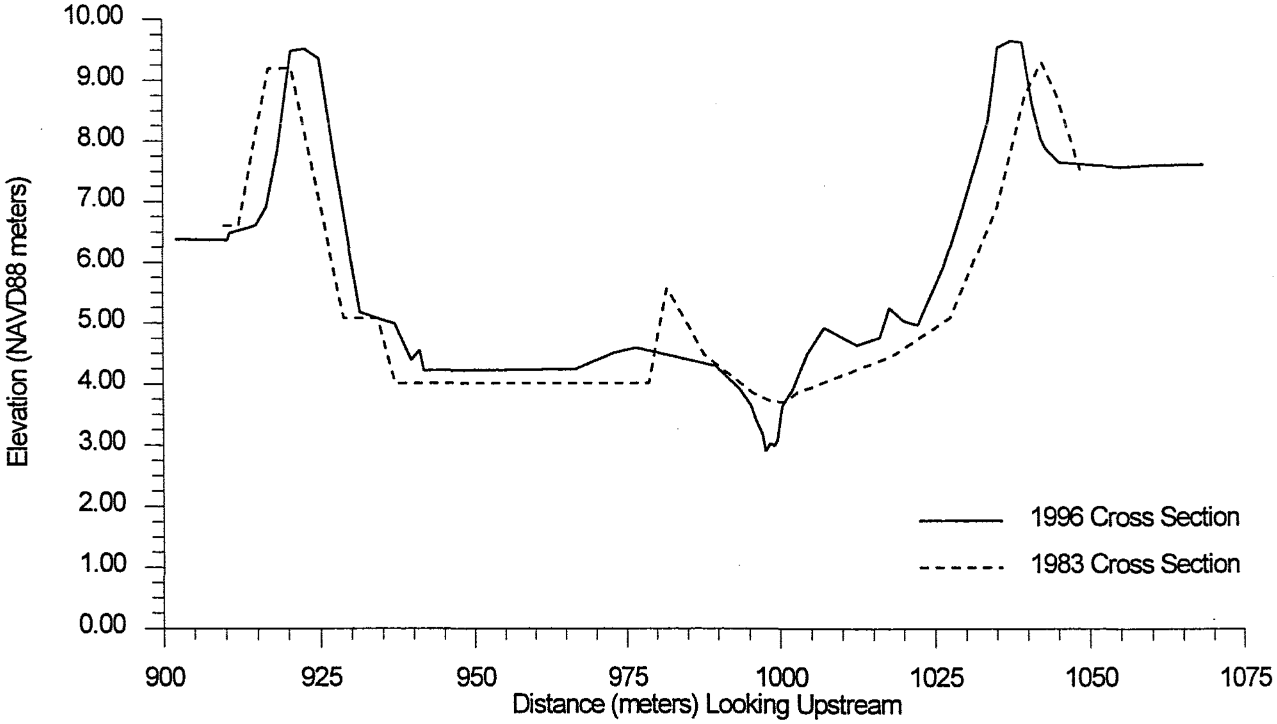

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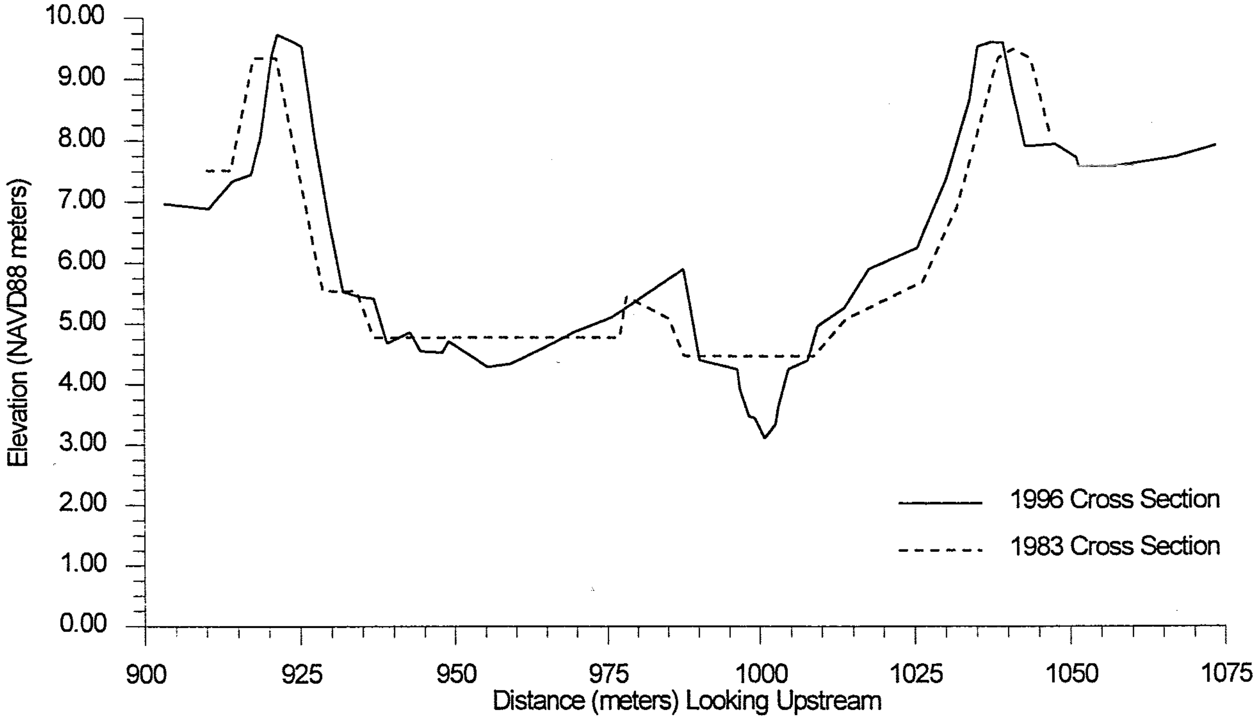

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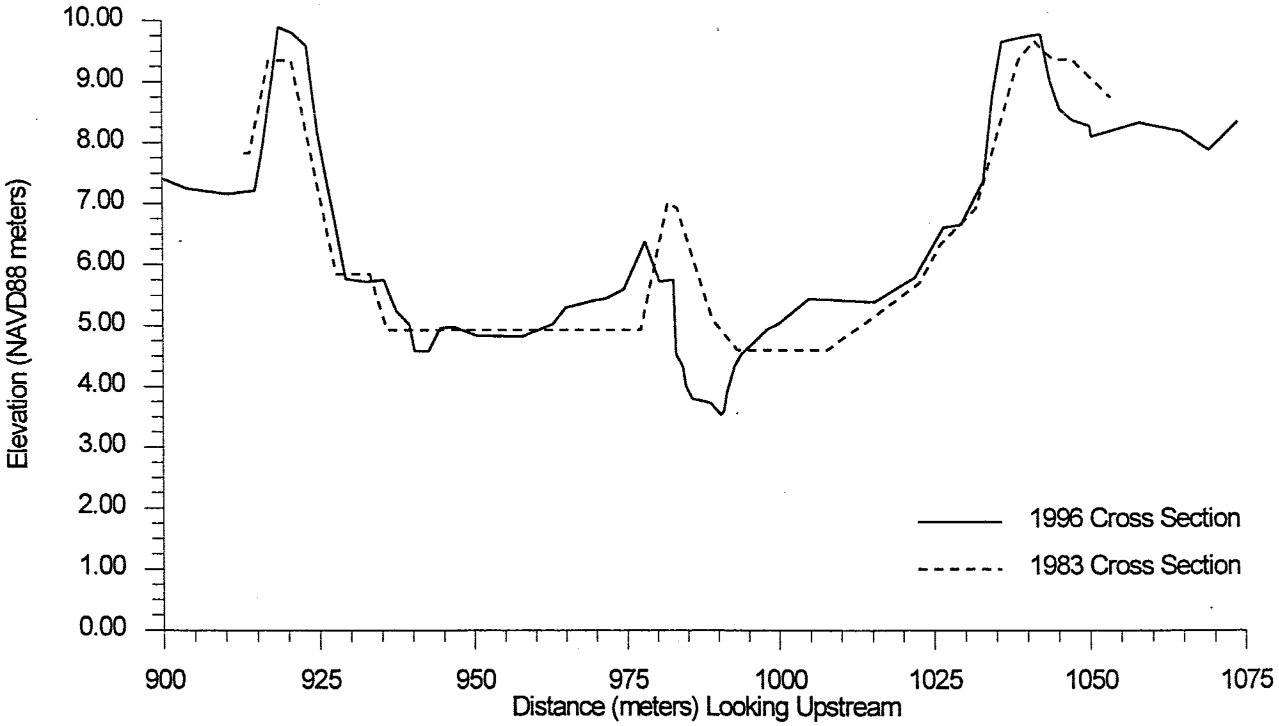

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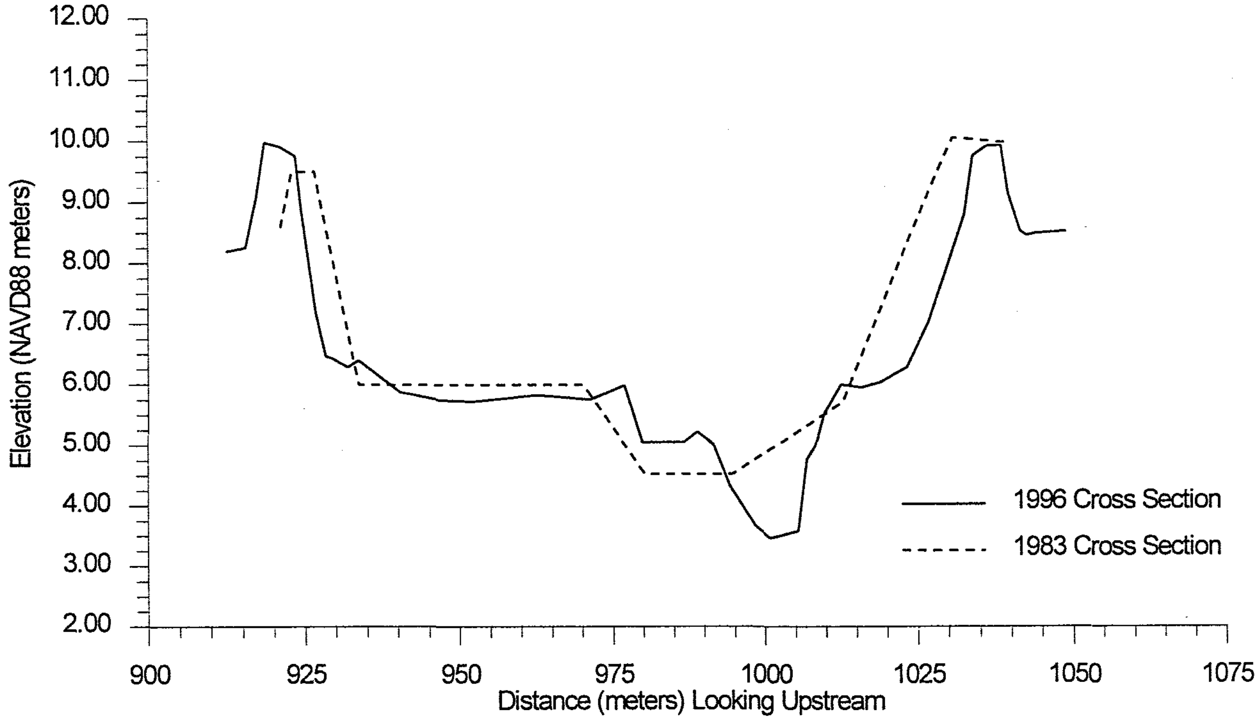

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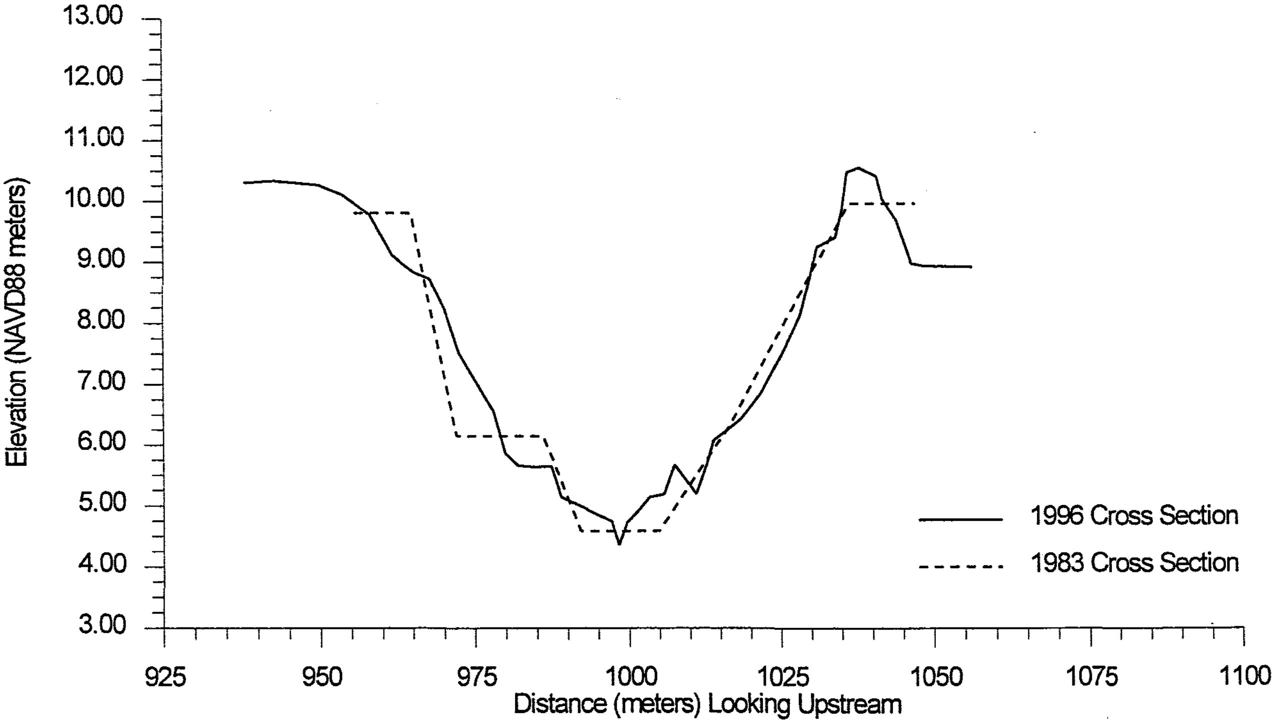

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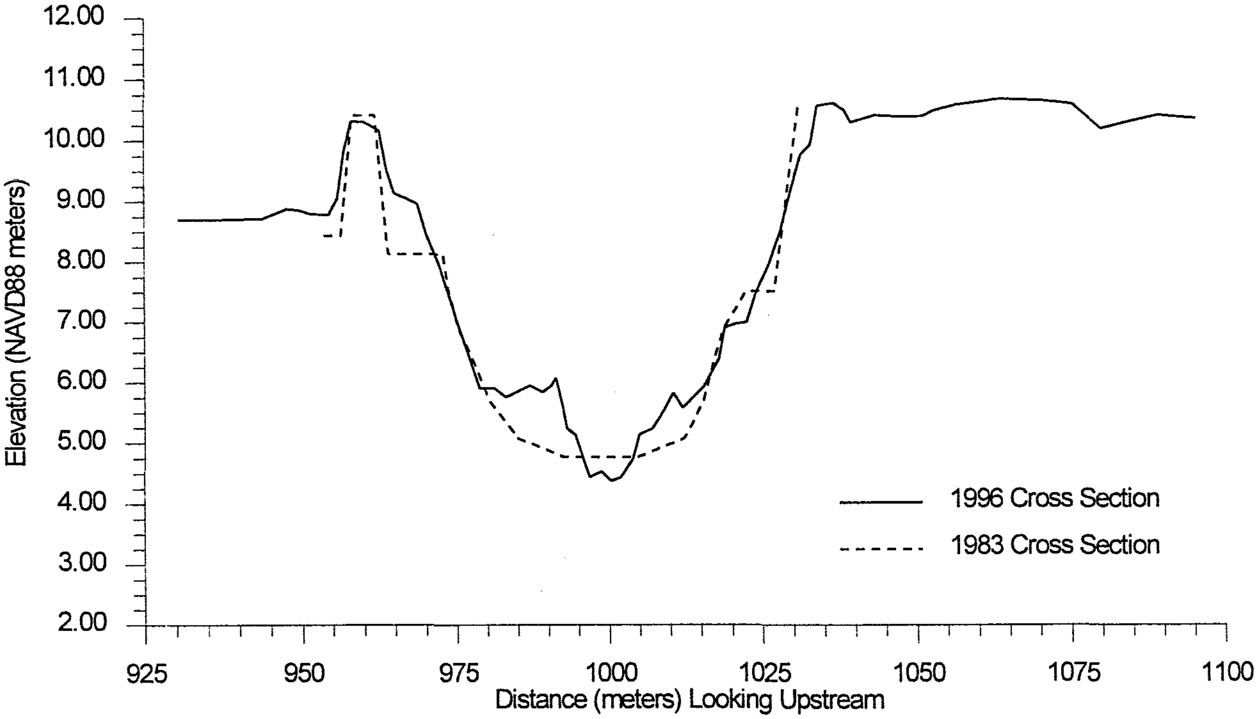

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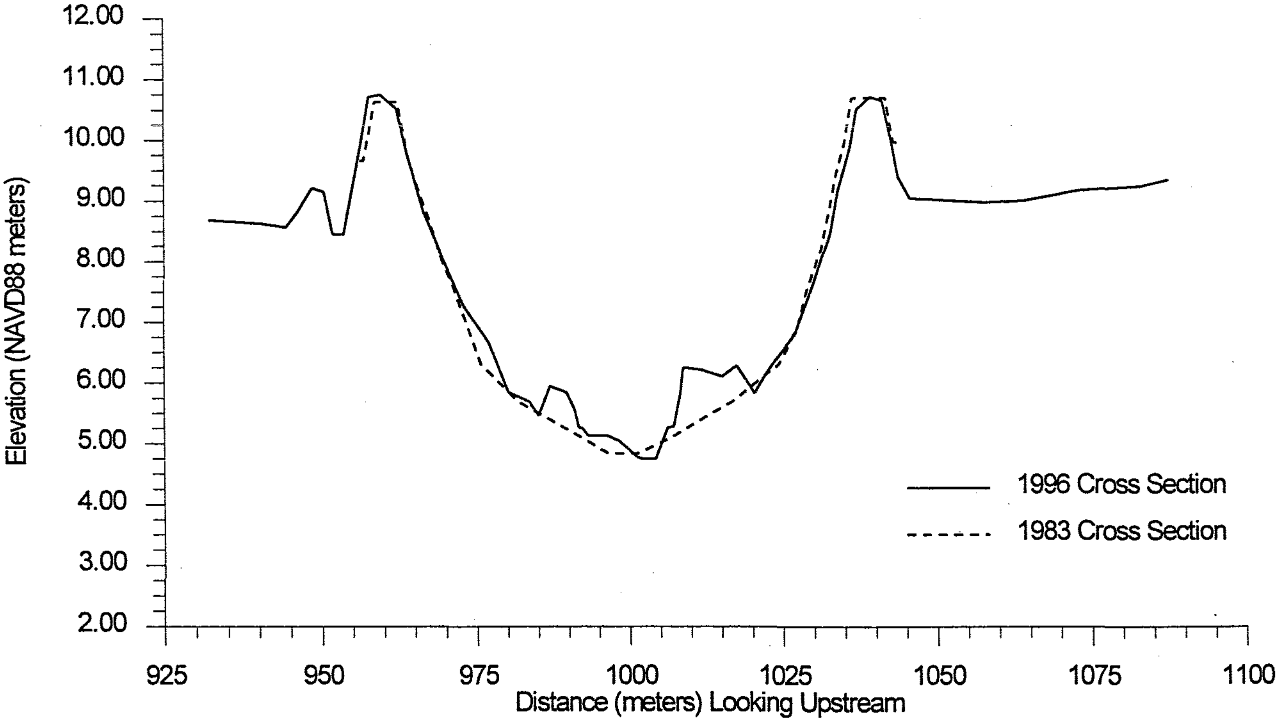

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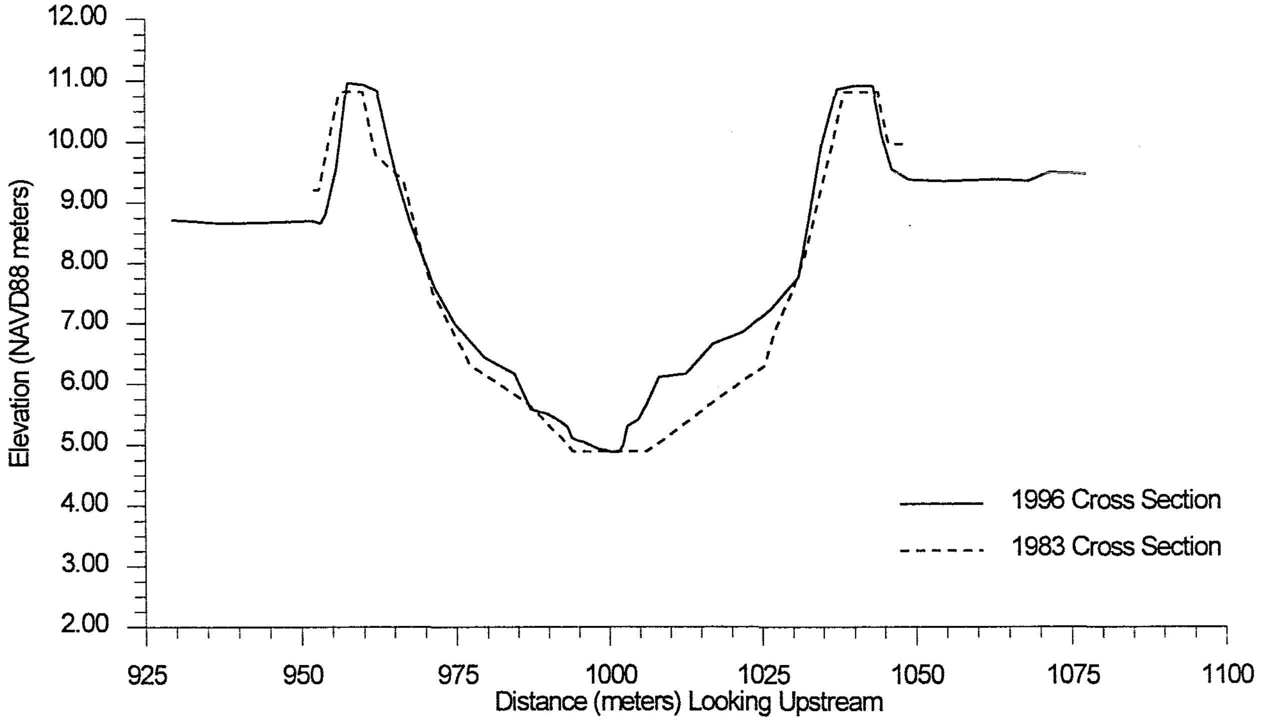

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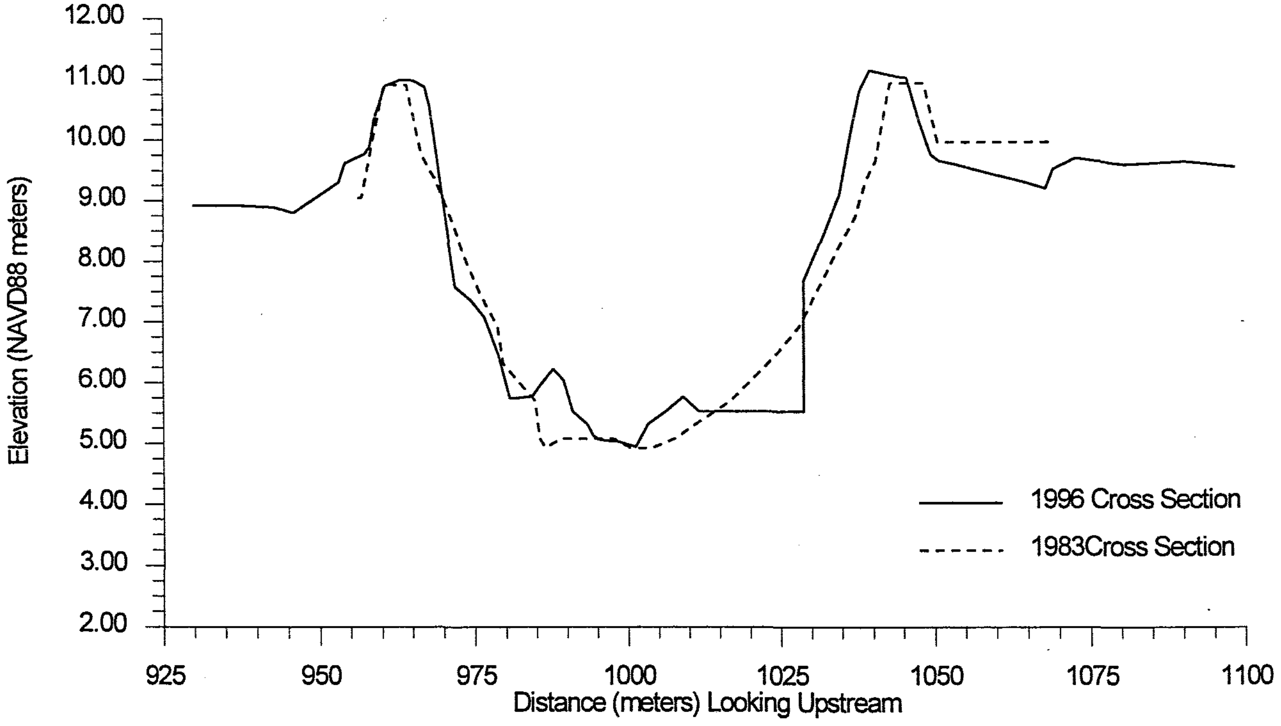

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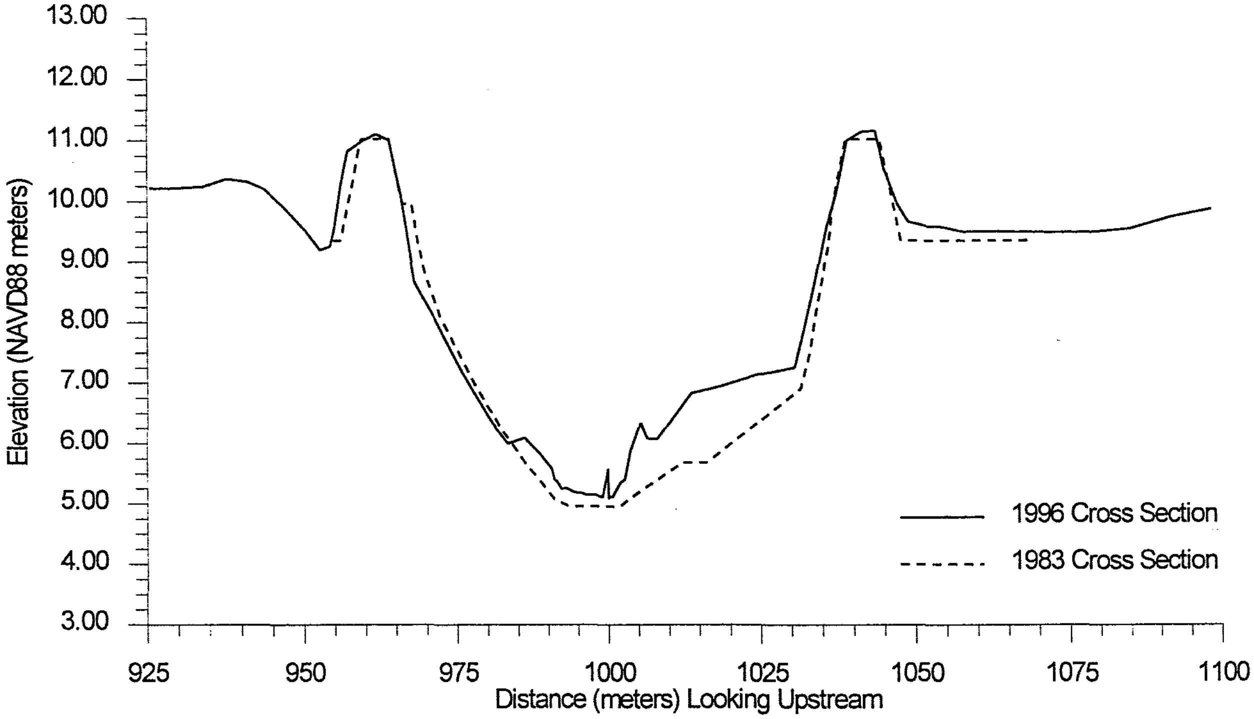

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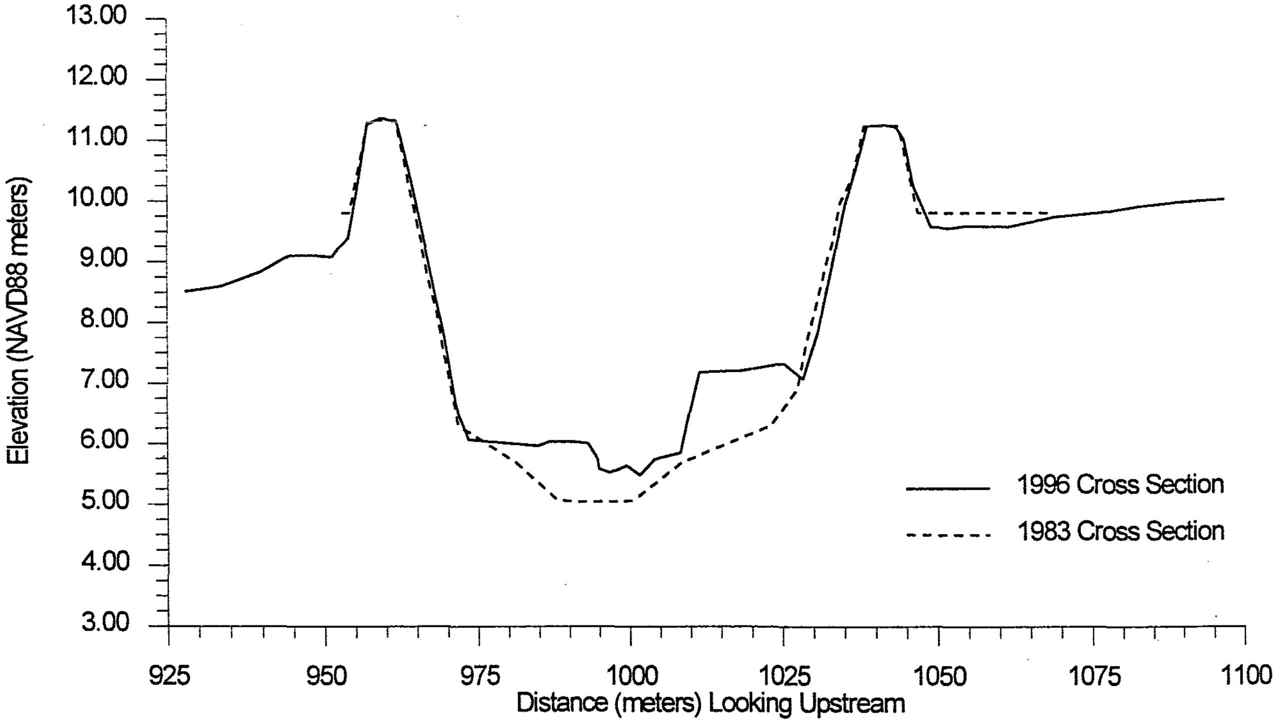
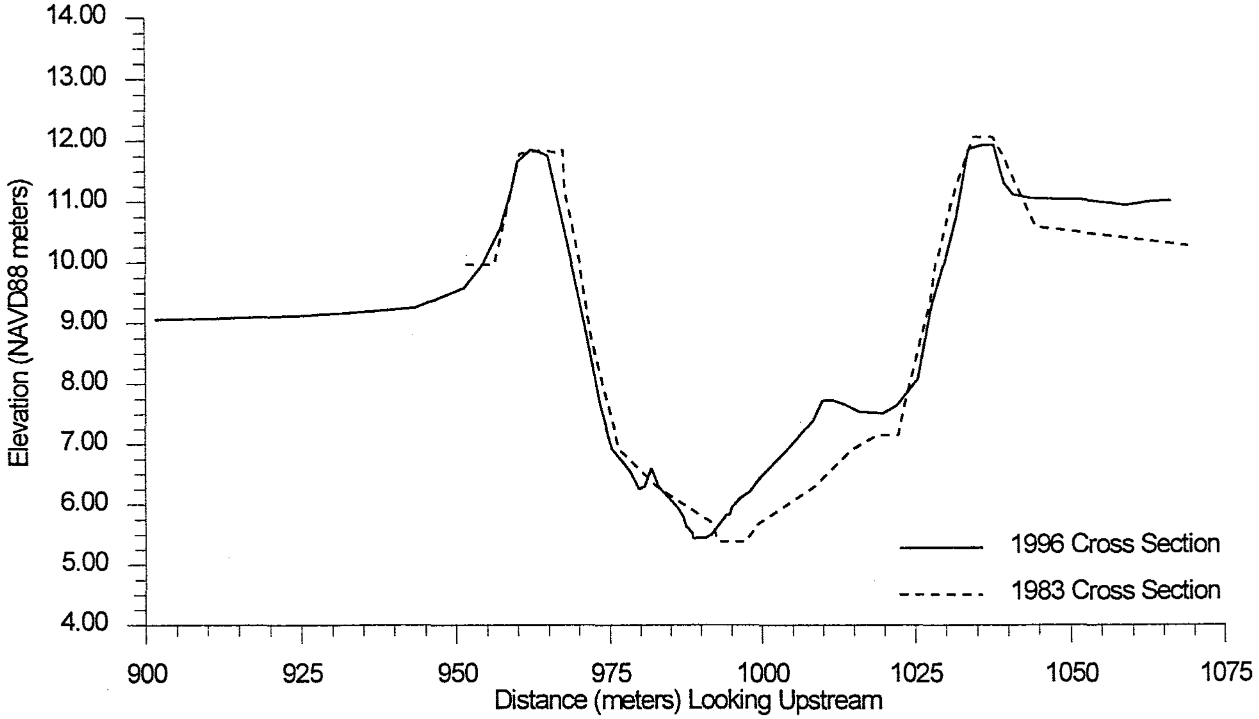


1996 Section 14220 (1983 Section 14225)



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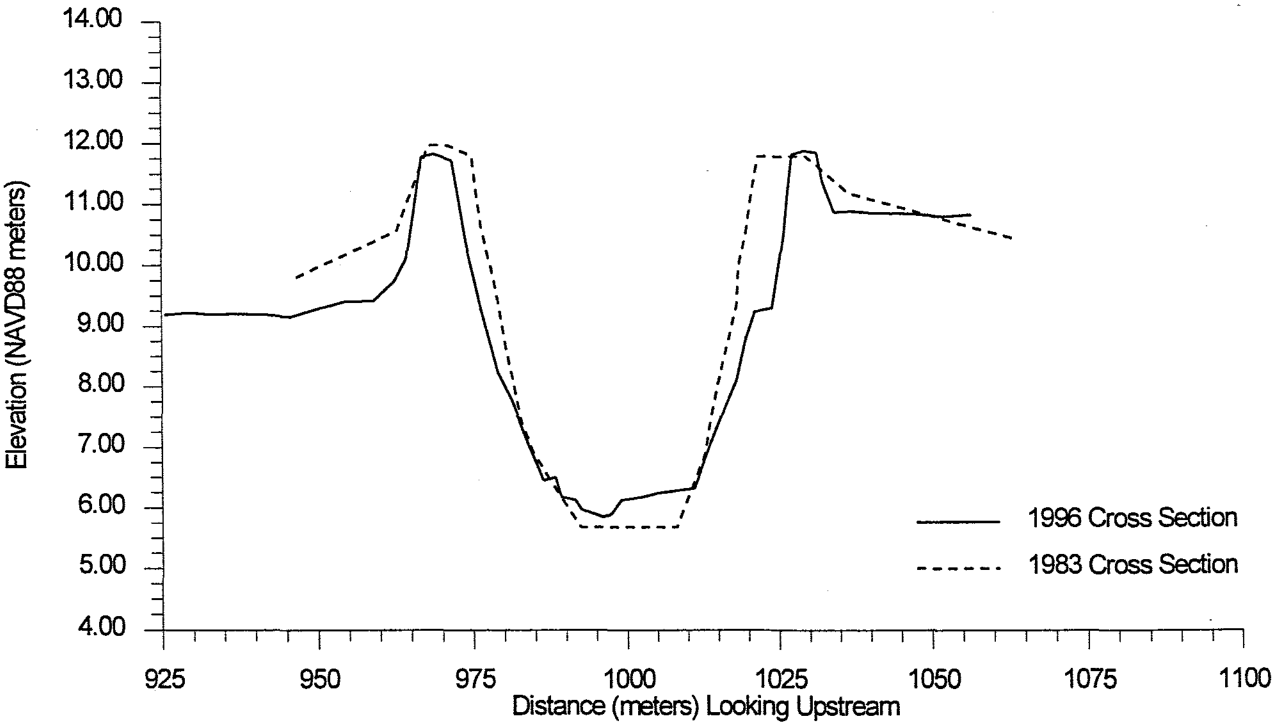
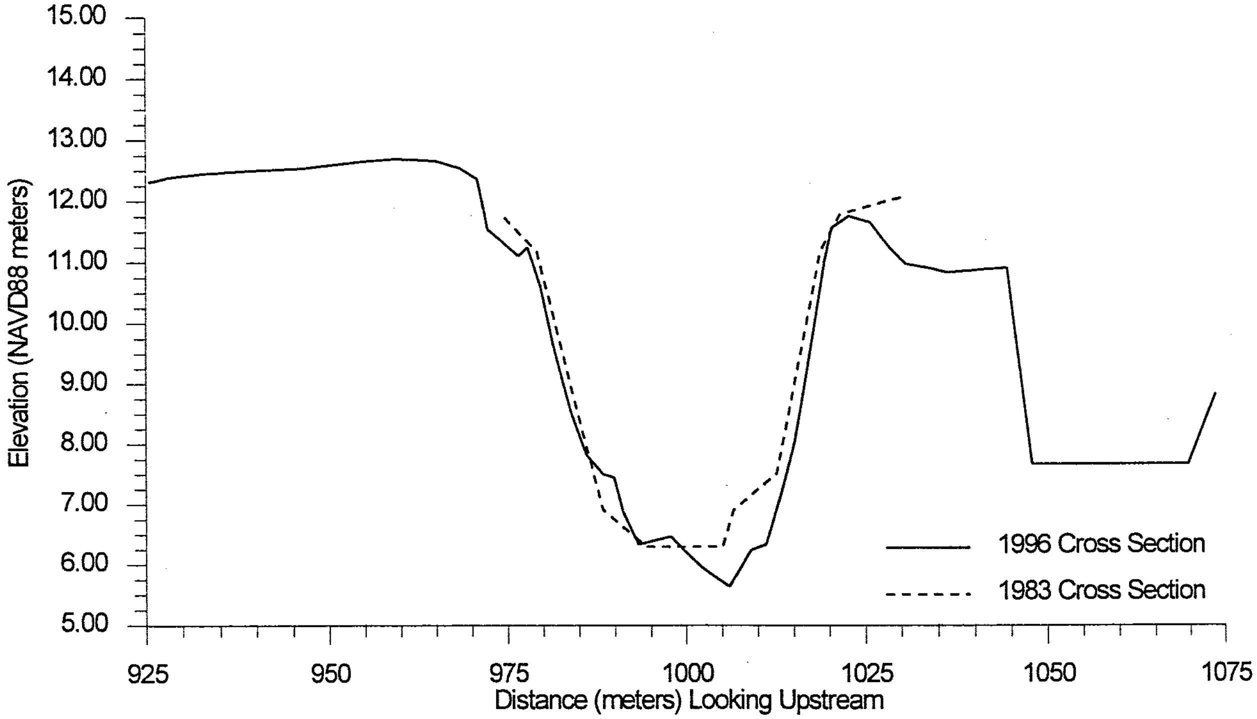


1996 Section 14460 (1983 Section 14507)



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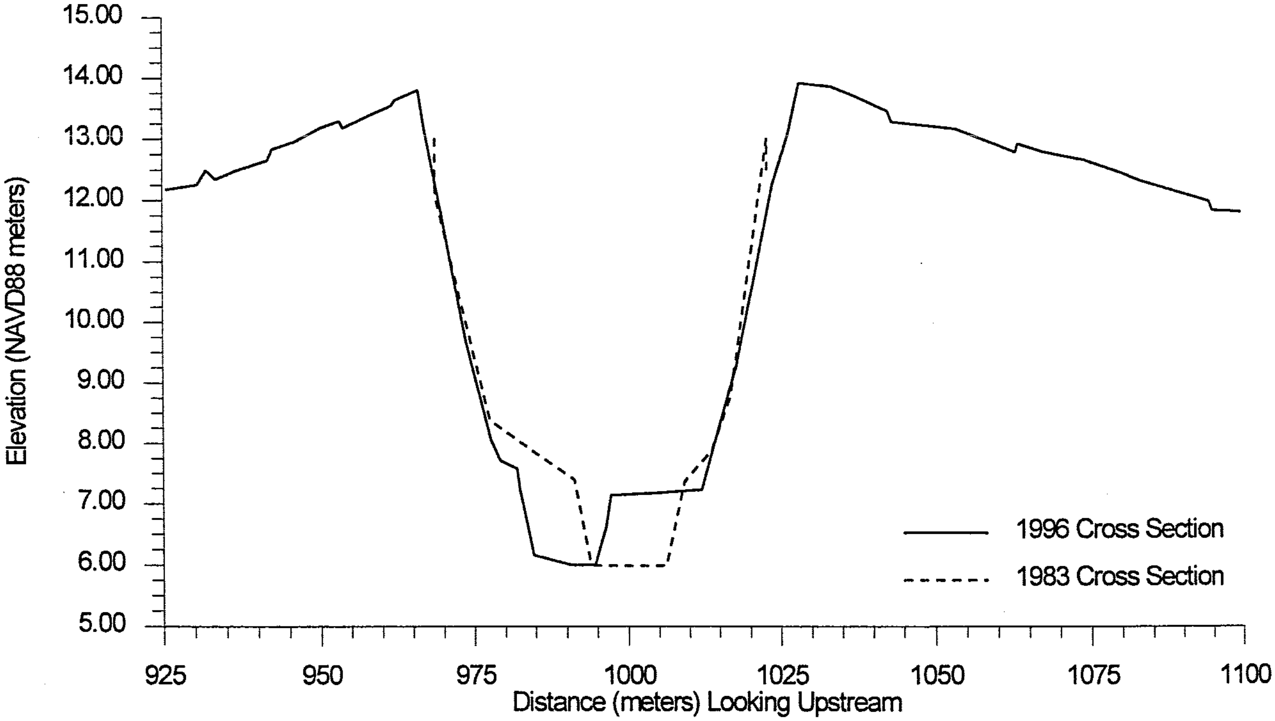
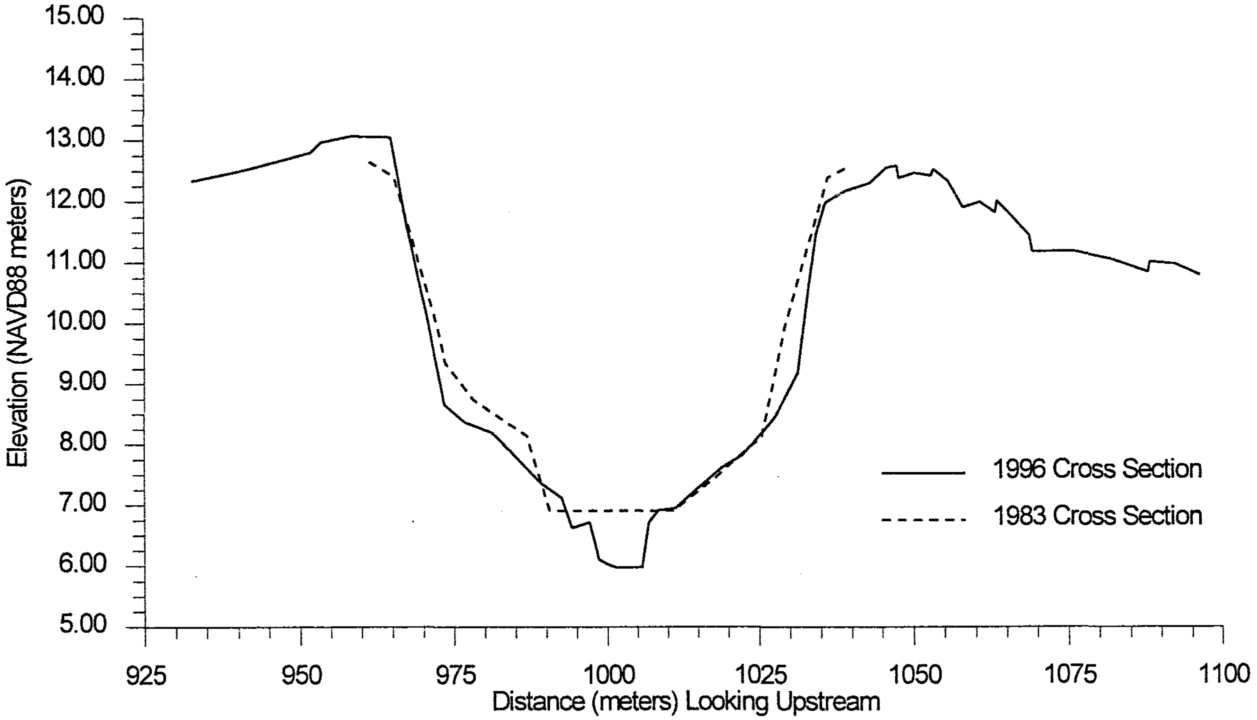


1996 Section 14687 (1983 Section 14711)



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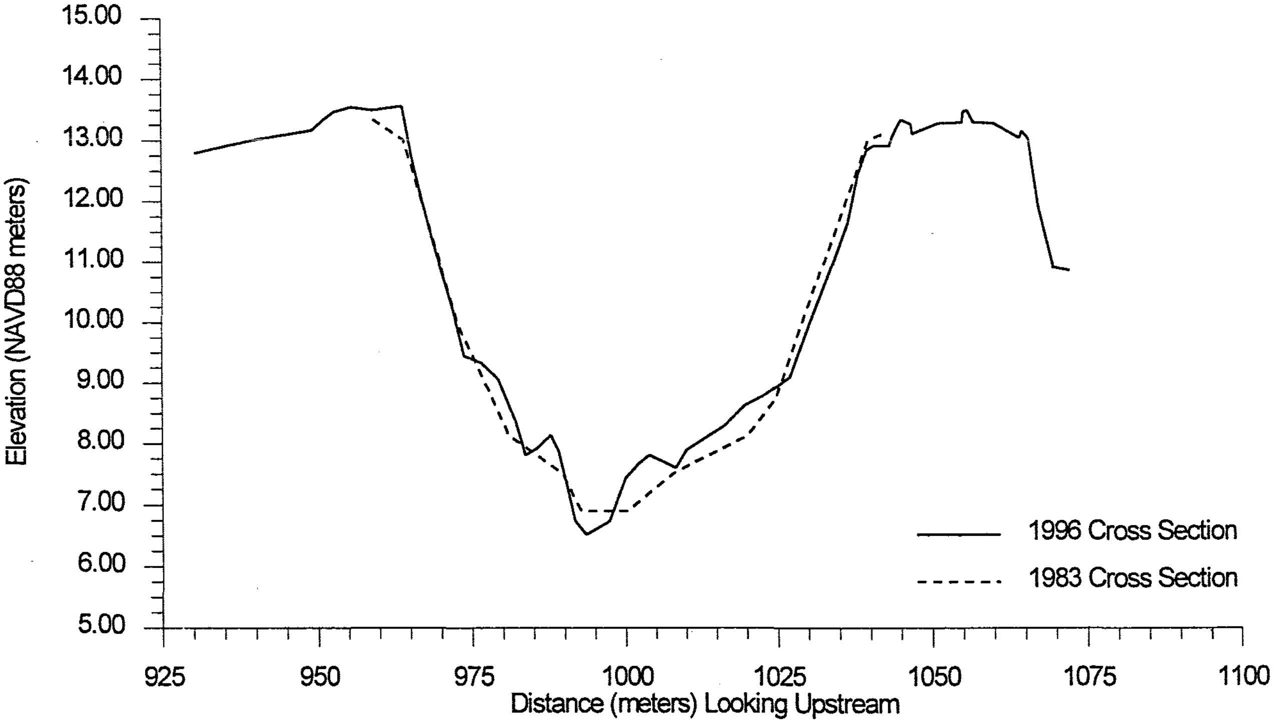
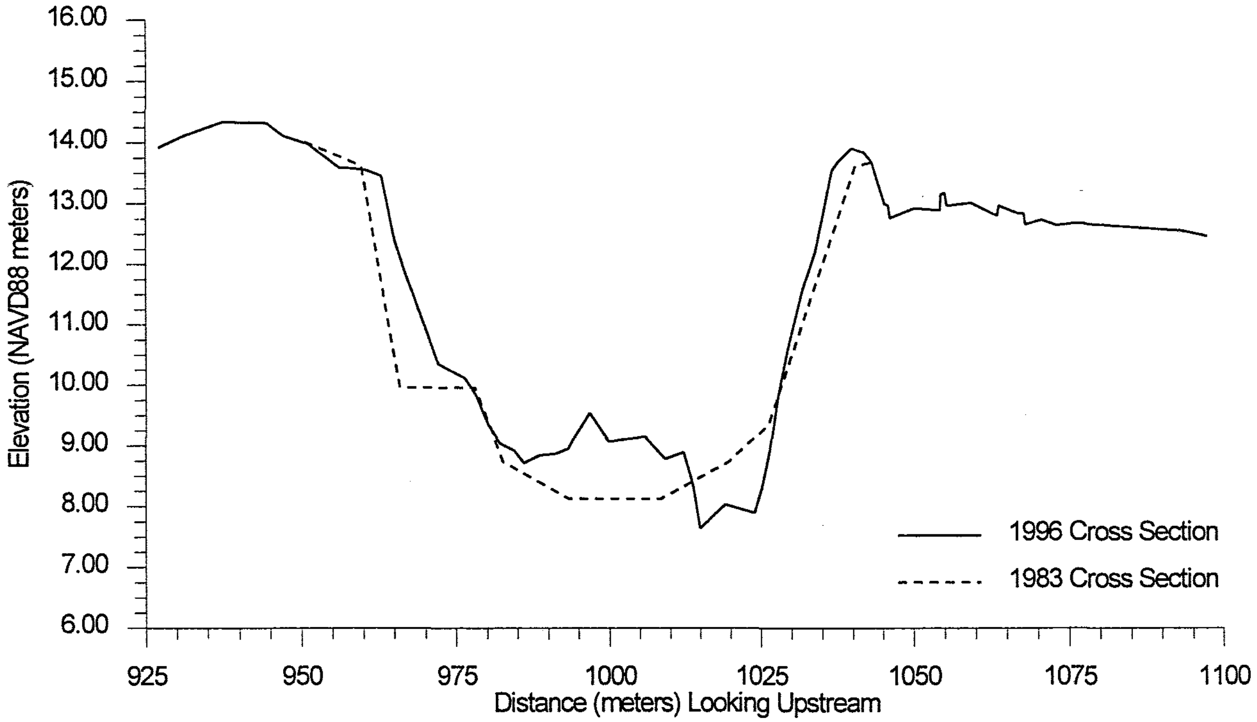
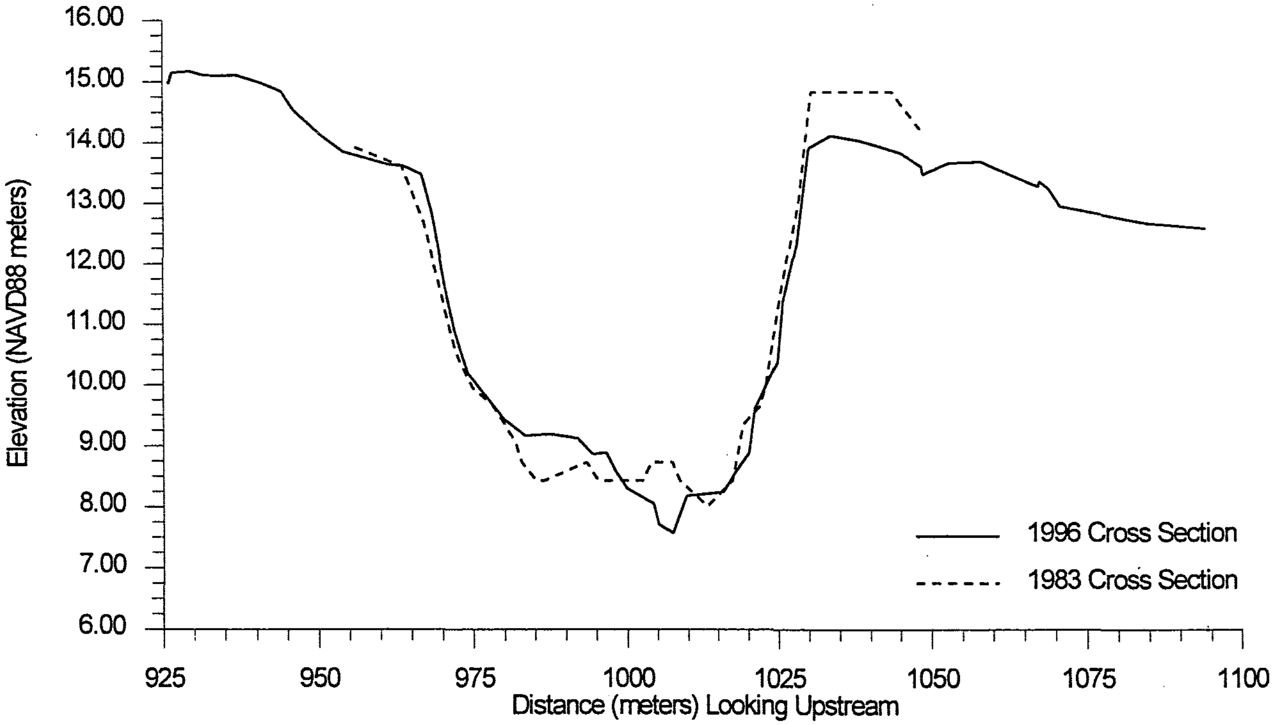


1996 Section 15176 (1983 Section 15210)



1996 Section 15570 (1983 Section 15586)



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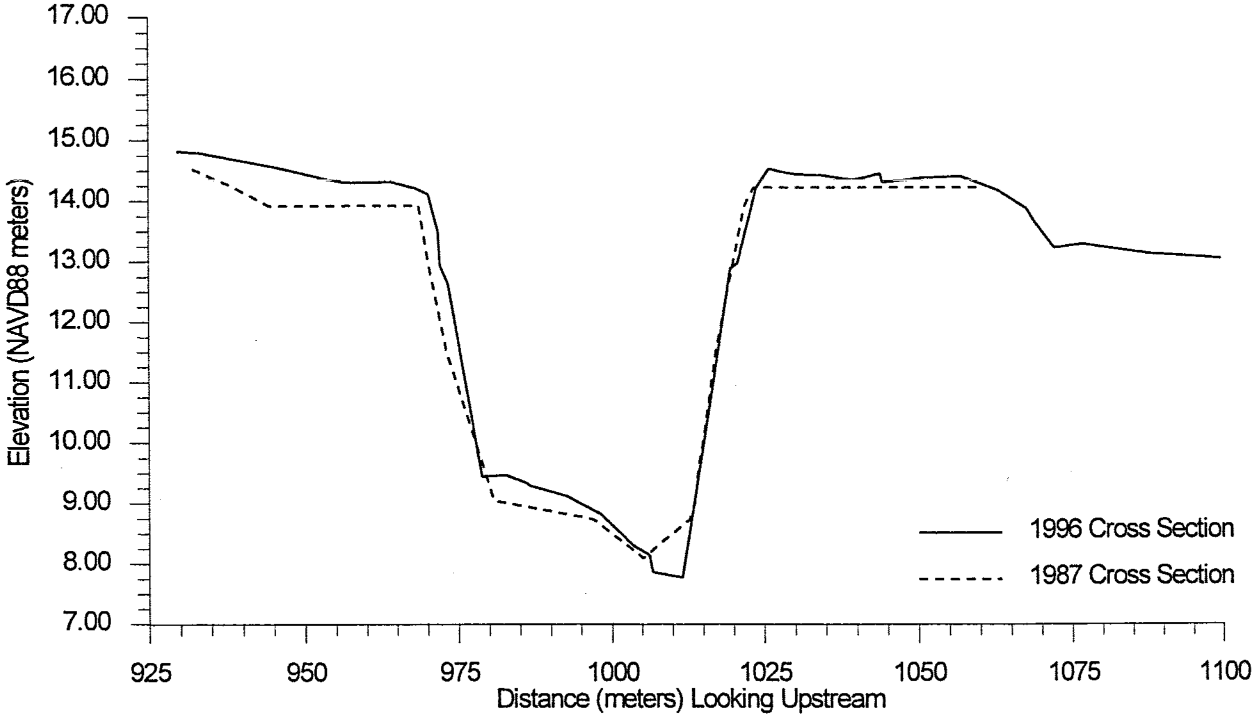

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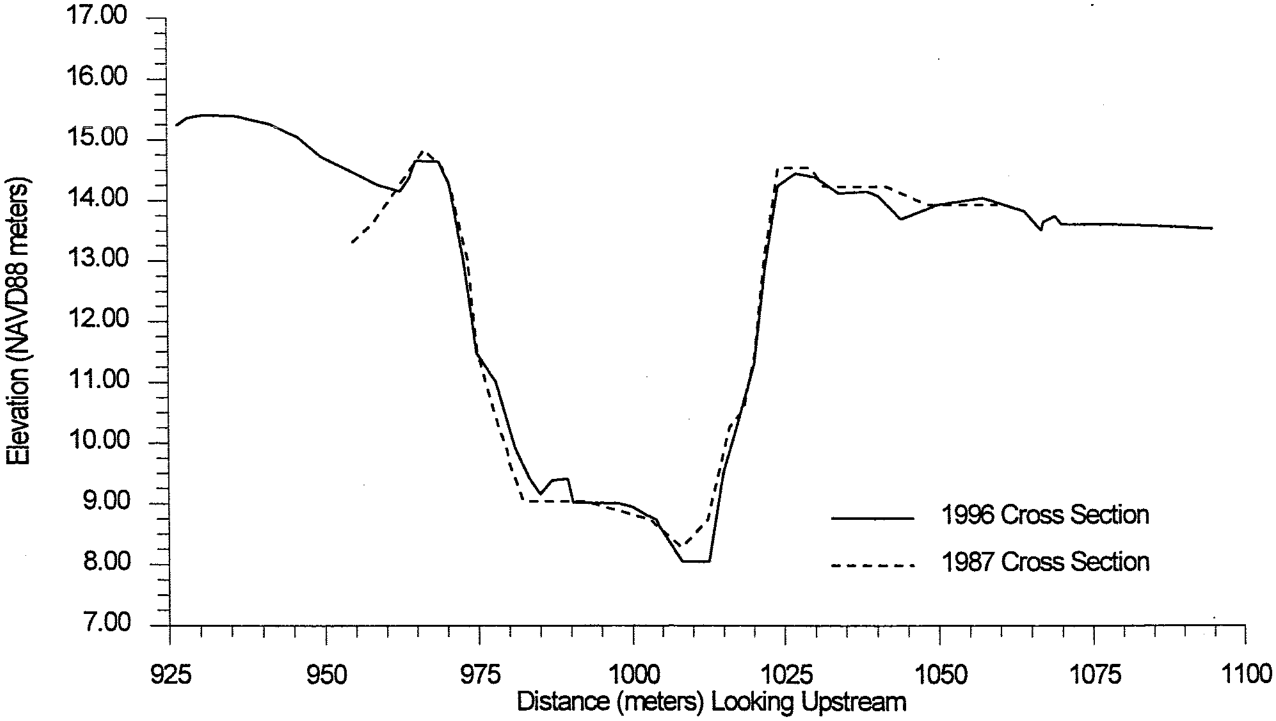

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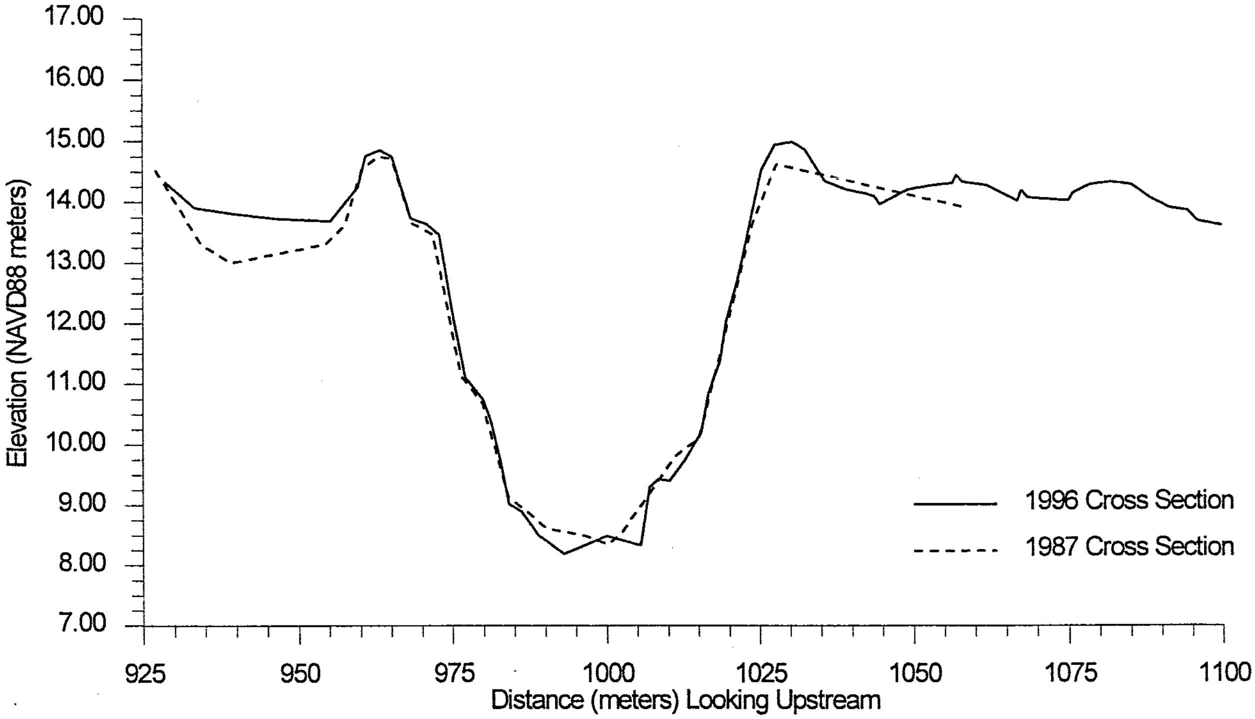

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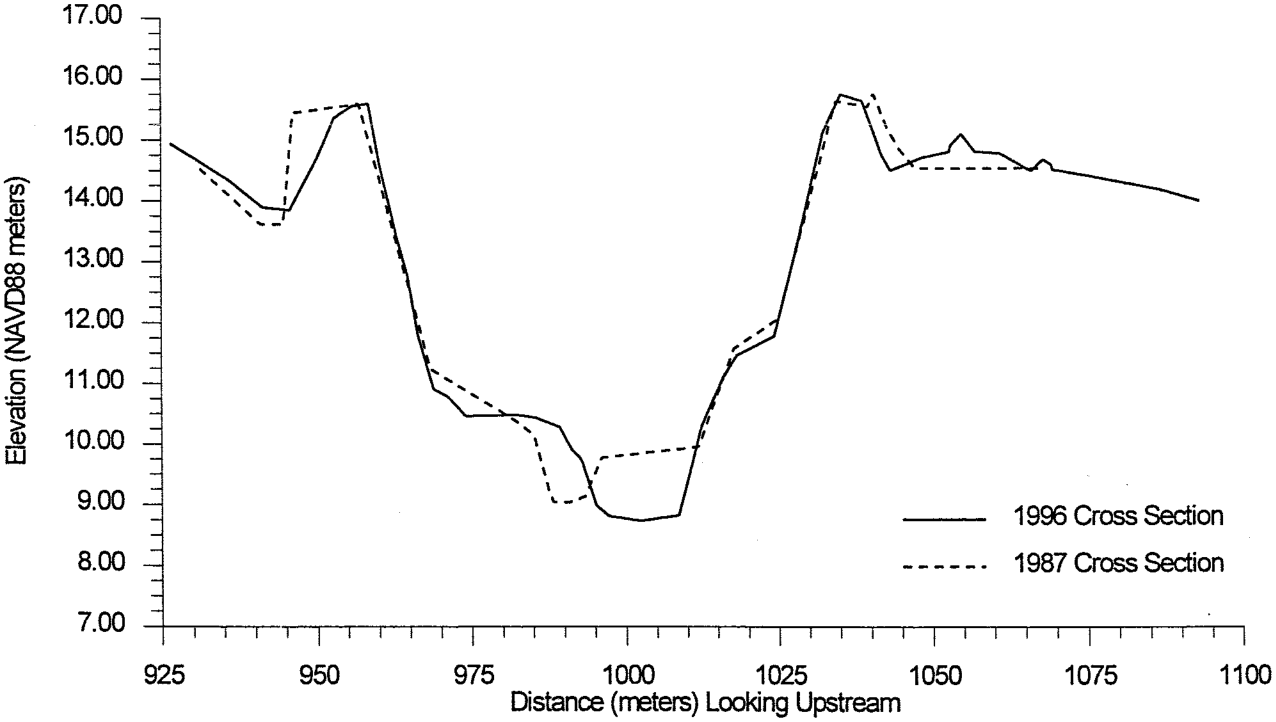
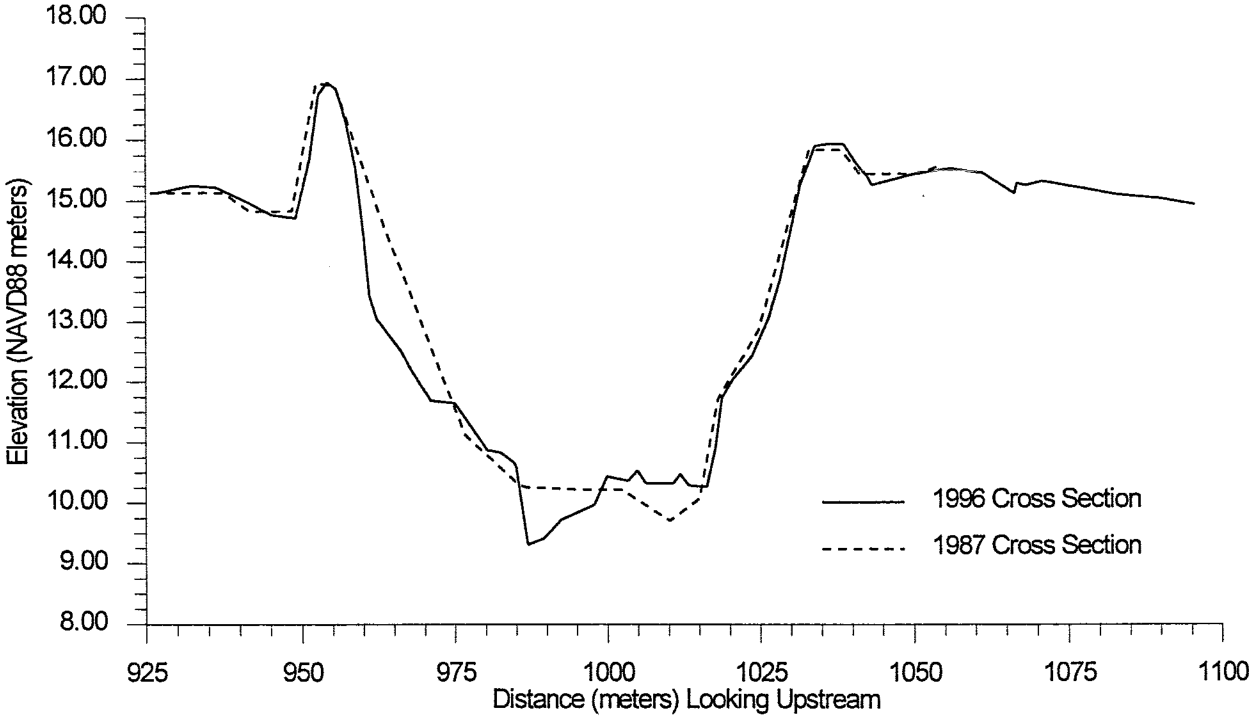


1996 Section 16110 (1987 Section 16153)



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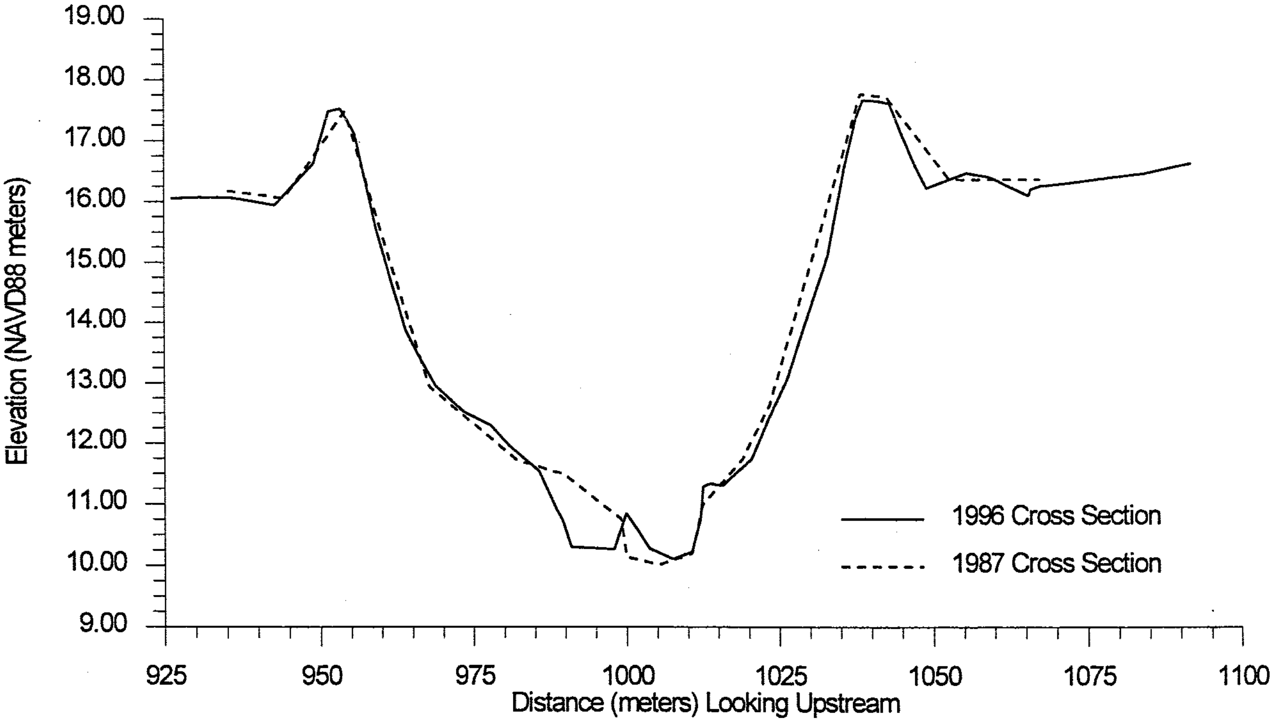

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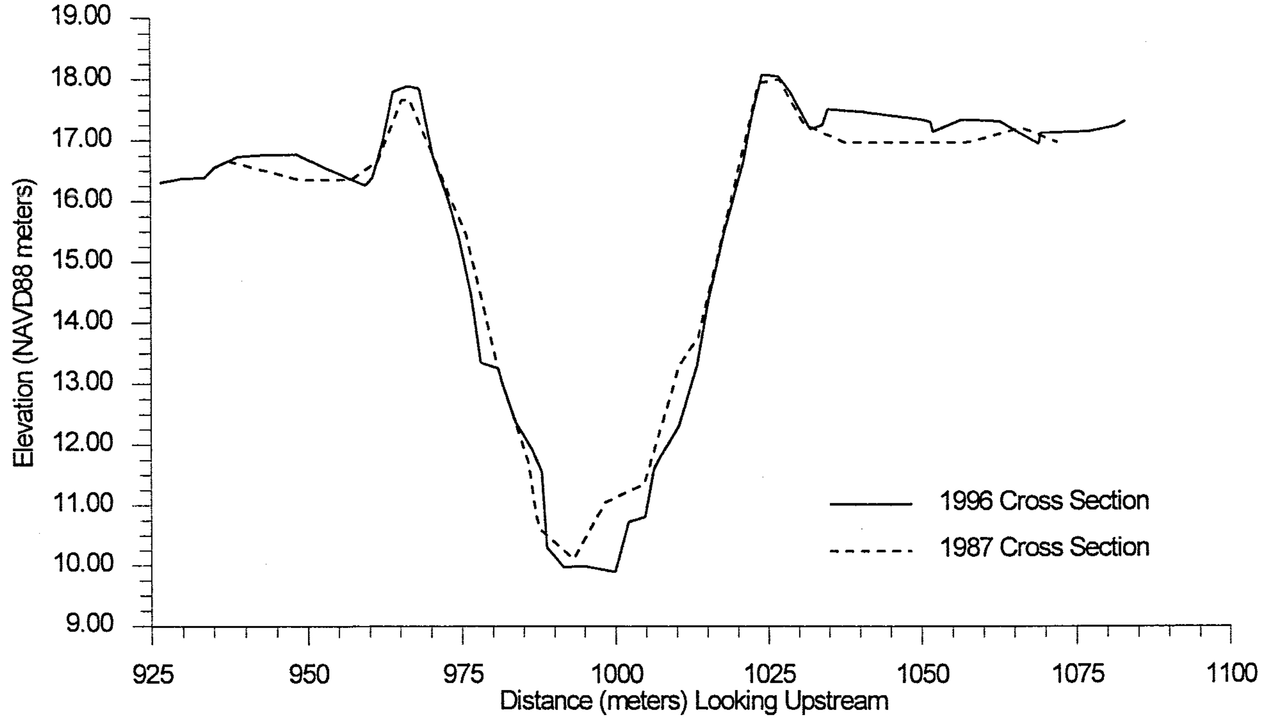
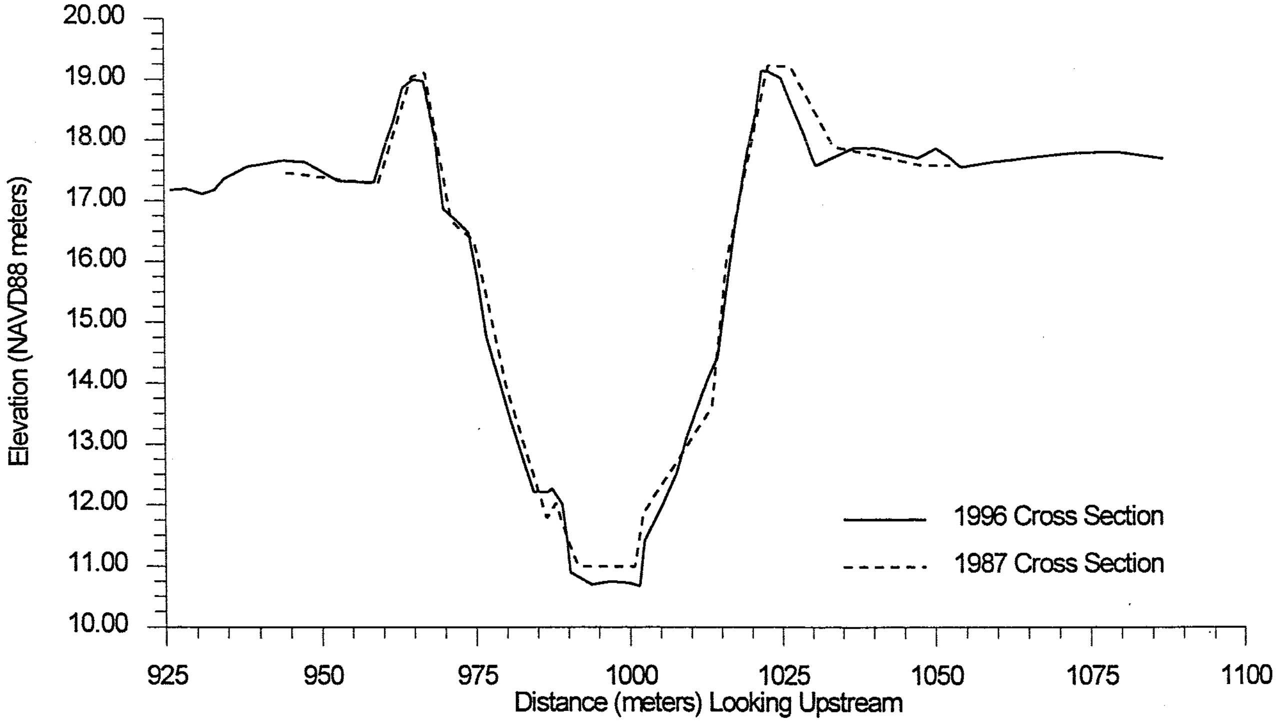
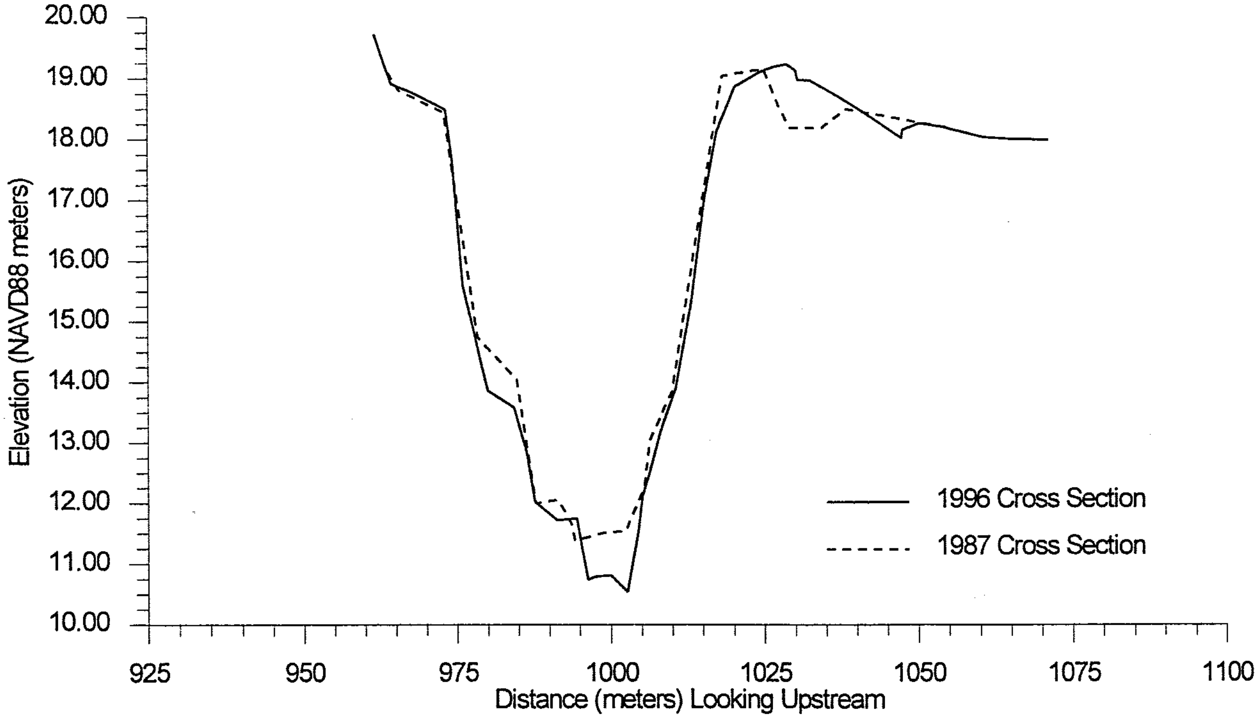


1996 Section 16980 (1987 Section 16995)



1996 Section 17250 (1987 Section 17286)



1996 Section 17490 (1987 Section 17516)

APPENDIX B

Laboratory Test Results for 20 Bag Sediment Samples

PARTICLE SIZE ANALYSIS BY HYDROMETER
(ASTM D422)

Particle Size	Sample No. 1	Sample No. 2	Sample No. 3
	(percent finer)	(percent finer)	(percent finer)
No. 10 Sieve	100	100	100
0.060 mm	98	100	93
0.050 mm	98	100	91
0.040 mm	98	100	89
0.030 mm	98	100	83
0.020 mm	97	96	77
0.010 mm	88	80	70
0.005 mm	79	71	62
0.002 mm	66	57	53
0.001 mm	53	46	40

PARTICLE SIZE ANALYSIS BY SIEVE
(ASTM D422)

Sieve Size	Sample No. 4	Sample No. 5	Sample No. 6	Sample No. 6b	Sample No. 7	Sample No. 8
	(% finer)	(% finer)	(% finer)	(% finer)	(% finer)	(% finer)
2 inch						
1½ inch			100	100		
1 inch			82	97	100	
¾ inch			72	83	97	
½ inch			56	71	92	
⅜ inch			45	62	85	
No. 4			21	40	65	
No. 10			9	20	44	100
No. 16	100	100	6	11	35	99
No. 30	98	98	4	6	22	98
No. 40	98	97	3	4	13	97
No. 100	95	93	1.4	2	2	90
No. 200	91.9	80.9	1.1	1.3	1.3	75.2

PARTICLE SIZE ANALYSIS BY SIEVE
(ASTM D422)

Sieve Size	Sample No. 9	Sample No. 10	Sample No. 11	Sample No. 12	Sample No. 13	Sample No. 14
	(% finer)	(% finer)	(% finer)	(% finer)	(% finer)	(% finer)
2 inch				100		
1½ inch				93		100
1 inch		100		83		94
¾ inch		99		76		89
½ inch		94		63		81
³/₈ inch		85	100	55		79
No. 4	100	54	99	38	100	76
No. 10	99	27	94	24	99	74
No. 16	98	17	86	17	98	73
No. 30	97	9	58	10	95	72
No. 40	97	6	32	6	88	70
No. 100	49	1.4	2	1	50	44
No. 200	23.3	1.0	0.8	0.8	33.5	27.0

Sieve Size	Sample No. 15	Sample No. 16	Sample No. 17	Sample No. 18	Sample No. 19	
	(% finer)	(% finer)	(% finer)	(% finer)	(% finer)	
2 inch		100		100	100	
1½ inch	100	89		91	98	
1 inch	93	80		68	92	
¾ inch	89	74		57	76	
½ inch	80	71		48	62	
³/₈ inch	70	69		42	54	
No. 4	50	64		31	33	
No. 10	33	60	100	20	15	
No. 16	25	57	99	13	10	
No. 30	18	52	96	7	5	
No. 40	12	48	95	4	3	
No. 100	2	28	76	1.3	1	
No. 200	1.4	19.4	52.8	1.0	0.6	

APPENDIX C

Laboratory Test Methods & Results for 5 Shelbly Tube Core Samples

7

RAY B. KRONE & ASSOCIATES
SEDIMENTATION • TIDAL HYDRAULICS

April 24, 1997

Dr. Robert C. MacArthur
Northwest Hydraulic Consultants
3950 Industrial Blvd., Suite 100C
West Sacramento, CA 95691-3430

Subject: Sampling and analysis of marsh sediment along Guadalupe Creek

Dear Dr. MacArthur:

Three core samples of the marsh soils at the downstream end, middle, and upstream end of Reach 0, their surface elevations, and their physical analyses are needed to calculate the effective suspended bay sediment solids content of the creek at those locations and to project the elevations of the marsh 100 years hence. This letter describes the procedures for obtaining the samples and for their analyses.

1. The samples should be taken where the marsh surface is level a short distance from the deeper channel bank. This distance is typically 10 to 20 feet, but may be further where waves have deposited material near the bank. The important consideration is that a level marsh surface be sampled.

2. The elevation (NGVD) of the marsh surface at the sample site should be determined to the nearest .01 ft.

3. The sampling tool should be at least 2-inches in diameter, and the sample should be retained in a metal or plastic tube and capped to retain moisture. The core length should be at least 2-feet and the core should include the marsh surface.

4. The sample should be refrigerated if analyses are made more than two days after sampling. For long storage, the sample should be held at 4 °C.

5. Analyses to determine bulk density, moisture content and organic matter profiles are required. Typically, a sample is slowly extruded from the retaining tube and 2-cm slices are produced. The average thickness of the slices is measured with a vernier caliper (the diameter of the retaining tube is noted), and the slices are placed in weighed ceramic dishes and the dish + sample weight to the nearest .01 gram is measured immediately to minimize evaporation.

a. The weighed samples are then placed in an oven that is maintained at 103 to 110 °C over night and placed in a dessicator to cool. The cooled sample + dishes are then weighed to determine moisture content. (16 hours is usually sufficient to remove moisture, but an initial set of samples should be cooled and weighed, reheated for 4 hours, cooled and reweighed to verify moisture removal. If a typical set indicate that removal is complete, then reheating the remaining samples is not necessary.)

b. The sample + dishes are then placed in a muffle at 600 °C and ashed to

P.O. BOX 694 • DAVIS, CA 95617 • TELEPHONE (916) 753-2555/752-8384

determine volatile solids (see Standard Methods). The ashed samples are cooled in a dessicator and weighed.

If the samples appear to be uniform along their length, economy of analysis can be realized by taking 2-cm slices along the top 10 centimeters then 10-cm slices along the remaining length of the core. If the core has thin layers of sand or organic matter the depth at which these layers should be noted by the laboratory technician.

The report of these analyses should include the sample location, the depth of the core analysed, and all measured weights.

The report of the marsh surface elevations should include the locations of the core samples and the elevations.

I look forward to these data and to modeling the marsh rise at these locations.

Sincerely,

Ray B. Krone

Ray B. Krone

Sunland Analytical Lab, Inc.
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 08/28/1997
Date Submitted 08/22/1997

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 8089. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %
39852	CORE 9+030	1	37.61	1.43	9.5
39853	CORE 9+030	2	39.11	1.34	10.4
39854	CORE 9+030	3	40.08	1.23	57.4
39855	CORE 9+030	4	49.73	1.11	12.3
39856	CORE 9+030	5	49.41	1.09	12.4
39857	CORE 9+030	6	51.21	1.00	11.6
39858	CORE 9+030	7	47.70	1.36	11.5
39859	CORE 9+030	8	49.84	1.01	10.9
39860	CORE 9+030	9	47.47	1.30	11.8
39861	CORE 9+030	10	37.12	1.31	8.8
39862	CORE 9+030	11	31.94	1.42	6.7
39863	CORE 9+030	12	44.49	1.26	10.5
39864	CORE 9+030	13	48.20	1.25	11.8

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11353 Pyrites Way, Suite 4
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To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 8090. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %
39865	CORE 9+030	14	49.01	1.16	12.8
39866	CORE 9+030	15	47.91	1.20	11.3
39867	CORE 9+030	16	46.53	1.17	9.7
39868	CORE 9+030	17	45.43	1.19	9.2
39869	CORE 9+030	18	30.99	1.77	4.9
39870	CORE 9+030	19	30.34	1.74	6.4
39871	CORE 9+030	20	29.60	1.53	27.4
39872	CORE 9+030	21	33.51	1.15	8.3
39873	CORE 9+030	22	46.15	1.00	15.4
39874	CORE 9+030	23	48.24	1.20	9.3
39875	CORE 9+030	24	43.83	1.10	8.5
39876	CORE 9+030	25	46.44	1.09	8.6
39877	CORE 9+030	26	51.12	0.85	9.9

Sunland Analytical Lab, Inc.
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 08/28/1997
Date Submitted 08/22/1997

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 8091. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %
39878	CORE 6+600	1	42.48	0.89	16.0
39879	CORE 6+600	2	39.29	1.27	13.0
39880	CORE 6+600	3	38.29	1.38	14.0
39881	CORE 6+600	4	39.37	1.22	14.3
39882	CORE 6+600	5	38.85	1.20	15.9
39883	CORE 6+600	6	40.61	1.25	12.9
39884	CORE 6+600	7	41.95	1.19	11.3
39885	CORE 6+600	8	40.48	1.31	12.5
39886	CORE 6+600	9	42.46	1.62	9.4
39887	CORE 6+600	10	39.82	1.17	10.4
39888	CORE 6+600	11	34.98	1.16	13.7
39889	CORE 6+600	12	29.65	1.04	18.9

Sunland Analytical Lab, Inc.
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 08/28/1997
Date Submitted 08/22/1997

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 8092. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %
39890	CORE 6+600	13	37.42	1.11	14.1
39891	CORE 6+600	14	37.60	1.15	15.3
39892	CORE 6+600	15	37.23	1.11	14.6
39893	CORE 6+600	16	38.93	1.28	17.7
39894	CORE 6+600	17	43.71	1.15	17.4
39895	CORE 6+600	18	42.68	1.19	12.2

northwest hydraulic consultants inc.

sacramento
vancouver
edmonton
seattle

3950 industrial boulevard, suite 100c
west sacramento, california 95691-3430
(916) 371-7475 - fax
(916) 371-7400

21 April, 1998

Dr. Ray Krone
R.B. Krone and Associates
PO Box 694
Davis, CA
95617

Subject: Guadalupe River Sedimentation Study - Bay Mud Samples

Dear Dr. Krone,

Please find enclosed the laboratory testing results for moisture, bulk density, and organic matter of the three bay mud samples we took earlier this year in February. Also enclosed are tables showing the depths of each sample and the ground surface elevation at each core site. Please note that all of these data are in **metric** units.

If you have any questions regarding these materials or require any additional information, please feel free to call at any time.

Yours Sincerely,
Northwest Hydraulic Consultants, Inc.

Rene F. Leclerc M.S.

Robert C. MacArthur, Ph.D., P.E.
Principal

Enclosures

RIVER ENGINEERING / HYDRAULIC MODEL TESTING / HYDRAULIC ANALYSIS AND DESIGN / COASTAL ENGINEERING
HYDROLOGY / SEDIMENTATION ENGINEERING / NUMERICAL MODELING / APPLIED RESEARCH / FORENSIC ENGINEERING

SCVWD

Land Surveying and Mapping

Project: Guadalupe River Soil Boring Sites
Folder: 121
Date Processed: 4/13/98
Horizontal Datum: NAD83
Vertical Datum: NAVD88
Note* All units in Metric Measurements

Point#	sta. (m)	Northing	Easting	Elevation	
P-3	0 m	607794.181	1864945.274	2.291	— CORE C
P-2	2100m	606048.177	1865810.633	2.191	— CORE B
P-1	4050m	605305.382	1867399.010	2.202	— CORE A

↑

DISTANCE UPSTREAM OF MOUTH

s:\Guad-rv\Fld121\SoilBoring.xls

Core A			Core B			Core C		
Sample Number	Average Thickness (cm)	Depth (cm)	Sample Number	Average Thickness (cm)	Depth (cm)	Sample Number	Average Thickness (cm)	Depth (cm)
A1	1.75	1.75	B1	2.34	2.34	C1	1.93	1.93
A2	1.83	3.58	B2	2.04	4.39	C2	2.27	4.19
A3	1.85	5.43	B3	1.95	6.33	C3	2.21	6.40
A4	1.88	7.31	B4	2.04	8.38	C4	2.29	8.69
A5	1.69	9.00	B5	2.09	10.46	C5	2.11	10.80
A6	1.72	10.72	B6	2.13	12.59	C6	2.12	12.92
A7	1.79	12.51	B7	2.18	14.77	C7	1.87	14.79
A8	1.89	14.40	B8	2.10	16.87	C8	2.20	16.99
A9	2.04	16.44	B9	1.95	18.82	C9	1.72	18.72
A10	2.04	18.47	B10	2.20	21.02	C10	2.21	20.92
A11	1.92	20.39	B11	2.01	23.03	C11	2.20	23.12
A12	1.95	22.34	B12	2.14	25.17	C12	2.21	25.33
A13	1.75	24.09	B13	2.00	27.17	C13	1.93	27.26
A14	1.69	25.78	B14	2.05	29.22	C14	2.03	29.30
A15	1.98	27.76	B15	1.97	31.19	C15	1.95	31.25
A16	1.53	29.29	B16	2.09	33.28	C16	2.37	33.62
A17	1.82	31.11	B17	1.75	35.03	C17	2.09	35.71
A18	1.66	32.77	B18	2.01	37.04	C18	2.06	37.77
A19	1.70	34.47	B19	2.07	39.12	C19	1.91	39.68
			B20	1.99	41.10	C20	2.17	41.85
			B21	2.18	43.28	C21	2.24	44.09
			B22	2.14	45.42	C22	2.03	46.12
			B23	2.12	47.54	C23	2.27	48.39
			B24	2.02	49.56	C24	2.27	50.67
			B25	2.13	51.69	C25	2.15	52.82
			B26	1.83	53.52	C26	2.29	55.11
			B27	1.94	55.46	C27	2.23	57.34
			B28	2.11	57.57	C28	2.12	59.46
			B29	1.94	59.51	C29	2.19	61.65
			B30	2.01	61.52	C30	2.03	63.68
			B31	2.19	63.71			
			B32	2.26	65.97			
			B33	2.03	68.00			
			B34	1.90	69.90			
			B35	2.01	71.91			

Sunland Analytical Lab, Inc.
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 02/12/98
Date Submitted 02/02/98

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 18679. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41328	CORE A	1 (TOP)	59.00	1.47	15.4	WATER +
41329	CORE A	2	53.13	1.31	13.5	ROOTS
41330	CORE A	3	53.93	1.28	14.0	ROOTS
41331	CORE A	4	54.72	1.29	13.9	ROOTS
41332	CORE A	5	53.91	1.25	15.2	ROOTS
41333	CORE A	6	55.93	1.32	13.5	ROOTS
41334	CORE A	7	54.24	1.38	13.5	ROOTS
41335	CORE A	8	53.58	1.24	13.6	ROOTS
41336	CORE A	9	52.17	1.19	13.2	
41337	CORE A	10	53.38	1.23	13.5	
41338	CORE A	11	54.34	1.32	12.4	
41339	CORE A	12	54.13	1.32	13.8	
41340	CORE A	13	53.19	1.31	13.2	DRK COLOR
41341	CORE A	14	54.58	1.29	13.1	DRK COLOR

Sunland Analytical Lab, Inc.
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 02/12/98
Date Submitted 02/02/98

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 18680. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41342	CORE A	15	52.95	1.32	13.4	DRK COLOR
41343	CORE A	16	53.44	1.34	13.0	DRK COLOR
41344	CORE A	17	52.77	1.34	12.9	DRK COLOR
41345	CORE A	18	54.04	1.48	12.8	DRK COLOR
41346	CORE A	19	55.03	1.40	11.9	DRK COLOR

Sunland Analytical Lab, Inc.
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 02/12/98
Date Submitted 02/02/98

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 18681. Your purchase order number is .
Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41347	CORE B	1 (TOP)	58.87	1.21	13.0	H20,ROOTS
41348	CORE B	2	56.35	1.14	13.6	
41349	CORE B	3	56.43	1.07	20.3	
41350	CORE B	4	55.47	1.19	13.0	
41351	CORE B	5	56.56	1.20	16.3	ROOTS
41352	CORE B	6	55.00	1.17	11.8	ROOTS
41353	CORE B	7	55.10	1.14	19.1	ROOTS
41354	CORE B	8	54.93	1.09	14.2	ROOTS
41355	CORE B	9	54.49	1.10	15.8	
41356	CORE B	10	55.76	1.11	14.5	ROOTS
41357	CORE B	11	55.77	1.25	15.0	
41358	CORE B	12	53.55	1.21	14.2	RTS, DRKE
41359	CORE B	13	54.62	1.14	15.1	ROOTS
41360	CORE B	14	54.27	1.17	14.8	

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To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 18682. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41361	CORE B	15	53.47	1.26	14.4	
41362	CORE B	16	54.19	1.25	13.4	ROOTS
41363	CORE B	17	55.01	1.12	14.0	
41364	CORE B	18	54.85	1.23	15.6	
41365	CORE B	19	55.99	1.19	15.2	ROOTS
41366	CORE B	20	57.66	1.12	15.8	ROOTS
41367	CORE B	21	58.79	1.05	16.7	DARKER
41368	CORE B	22	59.87	1.03	17.1	ROOTS
41369	CORE B	23	58.56	1.12	16.2	ROOTS
41370	CORE B	24	58.45	1.04	16.7	ROOTS
41371	CORE B	25	57.48	1.10	16.3	RTS.H20+
41372	CORE B	26	59.79	1.18	16.8	
41373	CORE B	27	56.84	1.19	16.8	
41374	CORE B	28	57.17	1.16	17.4	

Sunland Analytical Lab, Inc.
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Date Reported 02/12/98
Date Submitted 02/02/98

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 18683. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41375	CORE B	29	55.99	1.15	16.4	
41376	CORE B	30	57.18	1.23	15.3	ROOTS
41377	CORE B	31	57.36	1.12	15.1	
41378	CORE B	32	56.75	1.16	14.0	ROOTS
41379	CORE B	33	57.23	1.24	13.3	LTR,H20+
41380	CORE B	34	56.95	1.19	12.8	
41381	CORE B	35	54.91	1.23	12.4	

Sunland Analytical Lab, Inc.
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Date Reported 02/12/98
Date Submitted 02/02/98

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 18684. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41382	CORE C	1 (TOP)	53.75	1.17	17.7	ROOTS,H20
41383	CORE C	2	46.67	1.03	15.8	
41384	CORE C	3	46.13	1.05	16.5	ROOTS
41385	CORE C	4	47.22	1.08	16.8	ROOTS
41386	CORE C	5	48.06	1.14	14.8	
41387	CORE C	6	47.43	1.32	15.6	
41388	CORE C	7	48.93	1.13	14.8	
41389	CORE C	8	49.37	1.19	15.4	ROOTS
41390	CORE C	9	49.48	1.23	14.9	
41391	CORE C	10	49.59	1.19	15.9	ROOTS
41392	CORE C	11	48.95	1.30	14.5	
41393	CORE C	12	50.39	1.21	14.9	
41394	CORE C	13	50.36	1.25	16.4	RTS.DRKER
41395	CORE C	14	52.33	1.24	16.9	

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(916) 852-8557

Date Reported 02/12/98
Date Submitted 02/02/98

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 28685. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41396	CORE C	15	53.67	1.15	16.0	ROOTS
41397	CORE C	16	50.26	1.26	17.0	ROOTS
41398	CORE C	17	53.46	1.25	16.5	
41399	CORE C	18	52.94	1.27	15.7	ROOTS
41400	CORE C	19	54.27	1.14	15.0	RTS,H20+
41401	CORE C	20	52.22	1.21	15.1	ROOTS
41402	CORE C	21	53.75	1.20	16.3	RTS,DRKER
41403	CORE C	22	54.02	1.22	15.7	ROOTS
41404	CORE C	23	55.56	1.15	16.7	ROOTS
41405	CORE C	24	53.77	1.19	17.2	ROOTS
41406	CORE C	25	56.01	1.09	15.4	ROOTS
41407	CORE C	26	56.51	1.04	17.9	ROOTS
41408	CORE C	27	54.15	1.12	16.3	ROOTS
41409	CORE C	28	55.85	1.02	16.7	ROOTS

Sunland Analytical Lab, Inc.
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Date Reported 02/12/98
Date Submitted 02/02/98

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 38685. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41410	CORE C	29	54.68	1.21	15.8	ROOTS
41411	CORE C	30	54.84	1.28	16.2	ROOTS

APPENDIX D

Forcasts of Future Marsh Surface Elevations Along Alviso Slough

by

Ray B. Krone

CALCULATION OF EFFECTIVE SUSPENDED SOLIDS CONCENTRATIONS IN ALVISO SLOUGH, CALIFORNIA

R.B. KRONE
& ASSOCIATES

CALCULATION OF EFFECTIVE SUSPENDED SOLIDS CONCENTRATION IN ALVISO SLOUGH
Alviso, California

Prepared for

Northwest Hydraulic Consultants, Inc.
3950 Industrial Blvd., Suite 100
West Sacramento, CA 95691

by

Ray B. Krone & Associates
P.O. Box 694
Davis, CA 95617

June 3, 1998

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INTRODUCTION

Guadalupe Creek flows northward through the cities of San Jose and Alviso and discharges to South Bay through Alviso Slough at the southern tip of the San Francisco Bay system. The Creek and Alviso Slough are leveed where they pass through the cities and to its terminus in South Bay. The area enjoys a Mediterranean climate, with moderate warm dry weather during spring, summer, and fall. Rain is from frontal storms in winter. During the long dry period, the lower portion of the Creek and the Slough are subject to ebb and flood flows and changing water surface elevations that are driven by tides in the Bay.

South Bay is a very shallow bay having a mud bottom, and large mud flats are exposed during low tides. Daily breezes during spring and summer create waves on South Bay that suspend the sediment, and the suspended clay and silt particles are transported with tidal circulation into the contiguous sloughs. The broad marsh areas that the sloughs once drained are now blocked by levees, and the reduced upstream tidal prism caused reduced flows in the sloughs. The slow currents in the sloughs facilitate deposition of suspended particles.

The Santa Clara Valley Water District dredged an enlarged channel between the levees to -10 ft NGVD in 1963 to provide adequate conveyance for storm runoff. Sediment has deposited in the slough since that time, and cross-section plots now show a narrow channel with vegetated marsh margins between the channel edges and the levees. The channel conveyance is greatly reduced. The District seeks remedies for this continuing loss of conveyance and has contracted with Northwest Hydraulics Consultants for the development of numerical models for the evaluation of alternatives. A suspended sediment concentration that will produce the observed deposition rates is needed for a two-dimensional sediment transport model, and a prediction of future marsh surface elevations if no modification of the channel cross-section is made is needed. This study utilized a historical depth, the properties of four marsh cores, and the marsh elevation at the coring sites to provide an estimate of the effective suspended sediment concentration and to predict the long term elevations of the marginal marshes.

THE NUMERICAL MODEL

Tidal marshes that are built by accretion of suspended sediment are typified by a horizontal planar surface with a dendritic, serpentine channel structure. Photographs of the marsh planes at Stations 0+00 and 4+050 are presented in Figures 1 and 2. The temporal history of such deposits can be represented by the accumulation of sediment at a single location on the plane. A simple mass balance of suspended particles per unit area of bed at that location leads to a computation that can be made for long historical times in reasonable lengths of computer time. The mass balance used in this study is:

$$(Y_w - Y_m)\frac{dC}{dt} = -w_s C + (C_* - C)\frac{dY_w}{dt}$$

where

$$\begin{aligned} &C_* = C_0 \quad for \quad \frac{Y_w}{dt} > 0 \\ &C_* = C \quad otherwise \end{aligned}$$

and C is the suspended sediment concentration at the site, C_0 is the suspended sediment concentration in the flooding water, Y_w is the elevation of the water surface, Y_m is the elevation of the marsh surface, t is time, and w_s is the settling velocity of the suspended sediment. Clay and silt particles form aggregates in salt water, and the settling velocity is determined by the relative density and size of the aggregates. For a given aggregation scenario that has continued for some time, empirical data show that the median settling velocity can be represented by:

$$w_s = A C^{4/3}$$

where A is an empirical coefficient (110 for C in gm/cm^3 and w_s in cm/s). Substituting this relation in the mass balance equation makes the determination of the suspended solids concentration a function of time, the initial bed elevation, the tides, and the suspended sediment concentration in the flooding water.

In this application, all of the sediment that settles to the bed is assumed to remain. The rise of the bed elevation due to accumulation of depositing sediment during an integration time step, ΔY_m, is:

$$\Delta Y_m = \frac{A C^{7/3} \Delta t}{C_s}$$

The model requires historic tides, the initial bed elevation, bed subsidence, the dry mineral density of the sediment deposit, and the marsh surface elevation at a known time. The suspended sediment concentration in the in flowing water is adjusted until the model reproduces the measured surface elevation.

HISTORIC AND FUTURE TIDES

Records of tides at the Golden Gate are the longest tidal records in the United States. Mean sea level (MSL) at the Golden Gate has been rising at a nearly steady rate since 1929. A linear fit to the 1929 to 1991 data is shown in Figure 3. This line was used to extrapolate mean sea level into the future. (There has been much discussion in recent years about possible accelerated sea level rise rate due to global warming. In the absence of firm data, continuation of the recent trend to 2050 was selected for simulation of marsh rise to that time.)

Evaluation of recorded historical high and low tides by the National Ocean Survey showed that the range of the tide increased with rising sea level. Linear relations between mean high water (MHW) and the epoch mean sea level and between mean low water (MLW) and epoch mean sea level were found. The difference between mean sea level at the Golden Gate and that at Guadalupe Creek was maintained throughout the calculation. Tides during successive years were simulated by adjusting mean sea level to the measured or projected values and adjusting the mean range of the predicted 1993 tides by the linear relations. Cosine interpolation between high and low tides was used for bed elevations below mean tide level, and sine interpolation with a period of 12.41 hours was used for tides when the marsh rose above mean tide level. The model stepped through the tides of each year with 600 second (ten minute) time steps.

SEDIMENT DRY MINERAL DENSITIES

Four cores of the marsh plane along the lower 6.6 km of the channel were taken by Northwest Hydraulic Consultants, Inc., personnel. The moisture contents, bulk densities, and organic matter content (by ashing) were determined from slices of the cores by Sunland Analytical Lab, Inc. Their report is included in the Appendix. Marsh surface elevations at the coring sites were provided by the Santa Clara Valley Water District. Locations, elevations, and average bulk dry densities are presented in Table 1.

Table 1. Locations, Elevations, Densities, and Marsh Vegetation at Core Sites

Station, m from mouth	Elevation, m (1988 datum*)	Mineral Bulk Density gm/cu cm	Dominant Vegetation
0+00	2.291	0.2993	Pickleweed
2+100	2.191	0.3445	Pickleweed
4+050	2.202	0.3821	Mixed
6+600	2.08	0.558	Bullrush

* (Elevation, m - 0.81) 3.28084 = Elevation, ft NGVD1929

Station 0+00 is north of the catwalk to the transmission towers, and the marsh there would not have been dredged in 1963. The elevations at Stations 2+100 and 4+050 are nearly the same as that at Station 0+00, whereas the elevation at Station 6+600 is lower. The distance between the levees is also smaller at Station 6+600 than it is at the downstream stations. It is concluded that the material at Station 6+600 had accumulated since the site was dredged (1963), and that the sediment at the downstream sites had not been removed during the channel dredging.

Calculated dry mineral densities of the cores from the four stations are plotted in Figures 4 through 7. These plots all have the same scales so that the changes in density from station to station are apparent. The variability of dry densities shown in each figure is largely due to the varying amount of plant material in the marsh soil, and the increase in dry bulk density with distance upstream reflects the change in plant species with increasing fresh water influence. The markedly higher value (greater mineral content) for the core at Station 6+600 is due to the more rapid sediment accumulation that occurred after dredging.

Average values for each core were used in the computation of marsh rise above mean tide level -- the elevation at which Spartina begins to grow. There are no measurements of the properties of sediment below that level, needed for calculation of sediment accumulation after dredging at Station 6+600. A typical bulk density of 1.35 gm / cm^3 was used, having a calculated dry mineral density of 0.530 gm / cm^3. The dry mineral density of this deposit is expected to be lower than that of the marsh above because it was never desiccated; it consists of consolidated aggregates that deposited from suspension.

SUBSIDENCE

Groundwater pumping over years from numerous wells in the vicinity of San Jose dewatered aquacludes and caused subsidence of the ground surface. The maximum subsidence was 12.7 feet in downtown San Jose. Subsidence was stopped in 1973 by aggressive groundwater recharge. A graph of the distribution of ground surface subsidence in 1967 is shown in Figure 8 and a plot of the subsidence where the maximum was measured is shown in Figure 9 (Ref.1.) Figure 8 shows that the subsidence at the core sites was 4.1 ft in 1967. The annual subsidence based on the data in Figure 9, as a fraction of the 8 feet at the down town station, was prorated to the history shown in Figure 9, and the annual subsidence was subtracted from the calculated rise in bed surface elevation.

Ground surface subsidence has exacerbated flooding in the Alviso area.

CALCULATIONS AND FINDINGS

Calculations of sediment accretion at the three downstream stations began at year 1850 with the marsh elevation at mean high tide at that time. Simulation of the sediment surface elevations at Station 6+600 began at year 1963 and the dredged depth. The suspended solids concentrations in flooding water were adjusted during successive simulations at each station until the resulting marsh surface elevations were the observed ones. The actual suspended solids concentrations in bay and slough waters vary continuously as wind and water depth in South Bay vary. The suspended solids concentrations calculated by this model are those which, if they had existed continuously, would have created the observed marsh elevations. These are *effective* concentrations that are needed for application of a two-dimensional sediment transport model. The calculated concentrations are presented together with the elevations and mineral bulk densities in Table 2.

Table 2. Calculated Suspended Sediment Concentrations in Waters that Flood the Marshes.

Station, m from Mouth	Elevation, m (1988 datum)	Mineral Bulk Density gm/cu cm	Suspended Sediment Concentration, mg/l
0+00	2.291	0.2993	262
2+100	2.191	0.3445	209
4+050	2.202	0.3821	242
6+600	2.08	0.558	243
		Average:	239

The elevation and suspended solids concentration for Station 0+00 is expected to be a little higher than the values for the slough marshes because the site is exposed to wave overwash, which was not included in the model. There is no apparent explanation for the lower than average suspended solids concentration at Station 2+100, other than a possibly atypical mineral bulk density. The average concentration, 239 mg/l should be used in future simulations of sediment transport in Alveso Slough.

The calculated historical and future rise of the sediment surfaces at the mouth of the slough and that at Station 6+600 from the time that it was dredged, 1963, to the year 2050 are plotted in Figure 10. Each point on the plot is the elevation at the end of the year. The effect of ground subsidence on the marsh elevation at Station 0+00 is clearly apparent. The figure also shows that the rate of rise slows as the marsh surface rises above mean tide level and the frequency and duration of surface flooding diminish. Projected marsh elevations are presented in Table 3.

Table 3. Projected Marsh Soil Surface Elevations, meters

Station, m	Year 2025		Year 2050	
from Mouth	Elev., m	Elev. re MHW	Elev., m	Elev. re MHW
0+00	2.51	0.41	2.64	0.48
2+100	2.43	0.34	2.57	0.41
4+050	2.44	0.34	2.58	0.41
6+600	2.34	0.24	2.53	0.37

These projections are based on continuing rise of sea level at its rate during the last half century. If sea level rises at a higher rate than projected, the marsh elevation will rise faster than projected.

REFERENCE

Poland, J. F., and Ireland, R. L., Land Subsidence in the Santa Clara Valley, California as of 1982, U.S. Geological Survey Prof. Paper 497-F, U.S. Gov't Printing Office, Washington, DC, 1988.

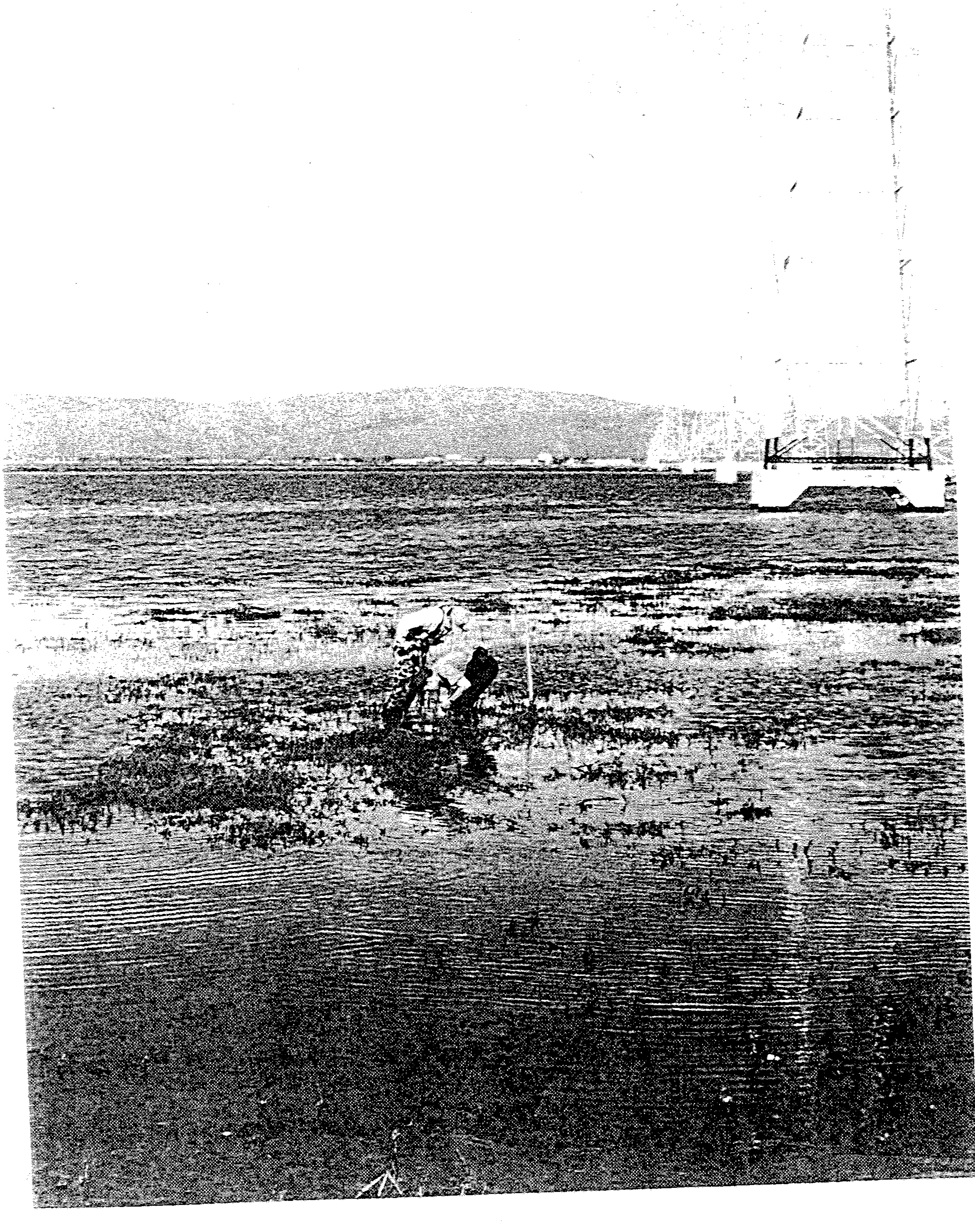


FIG. 1, MARSH AT STATION 0+00

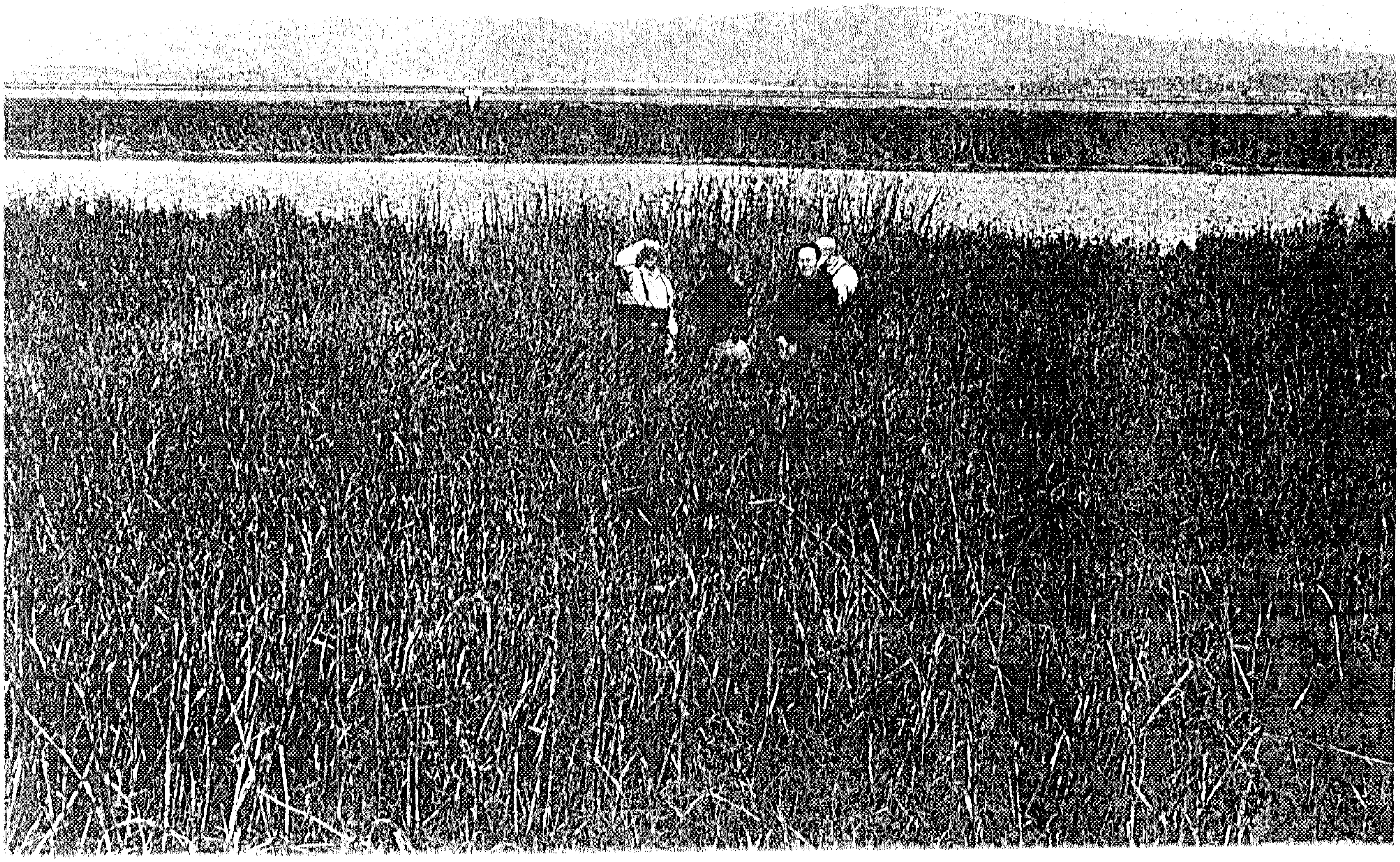


FIG. 2, MARSH AT STATION 4+050

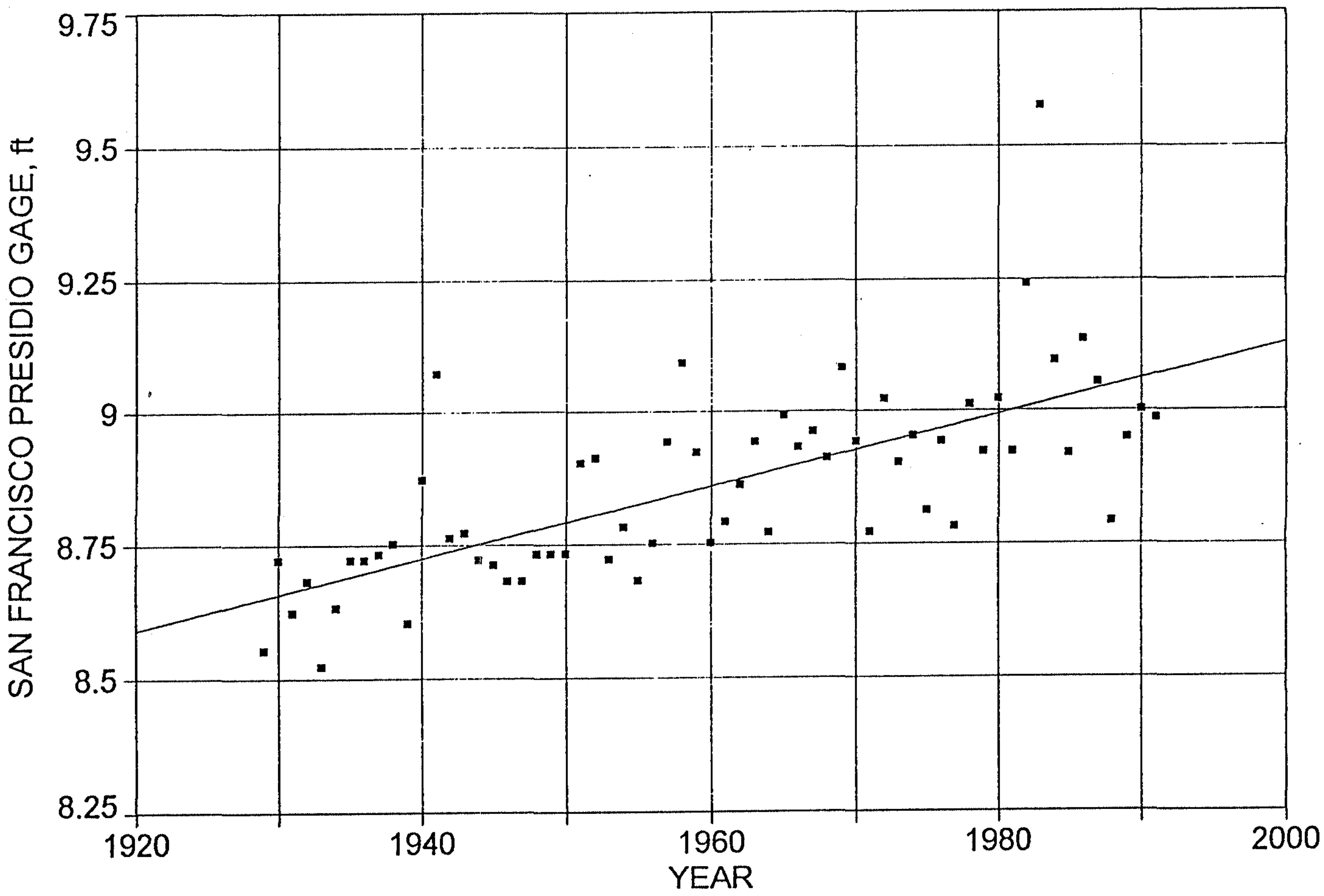


FIG. 3, LINEAR FIT TO POST-1929 MEAN SEA LEVEL DATA

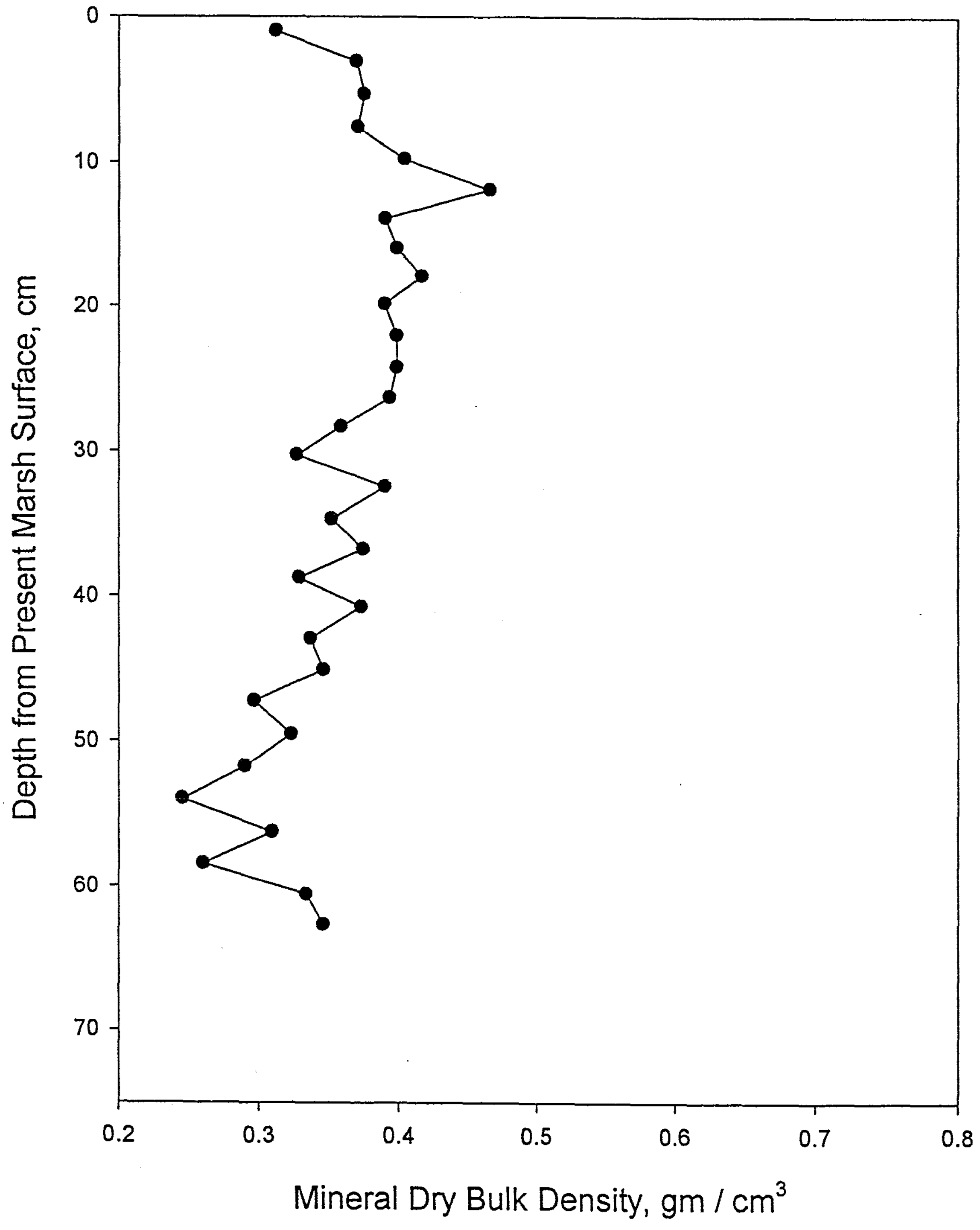


FIG. 4, MINERAL DRY BULK DENSITIES OF CORE
FROM STATION 0+00

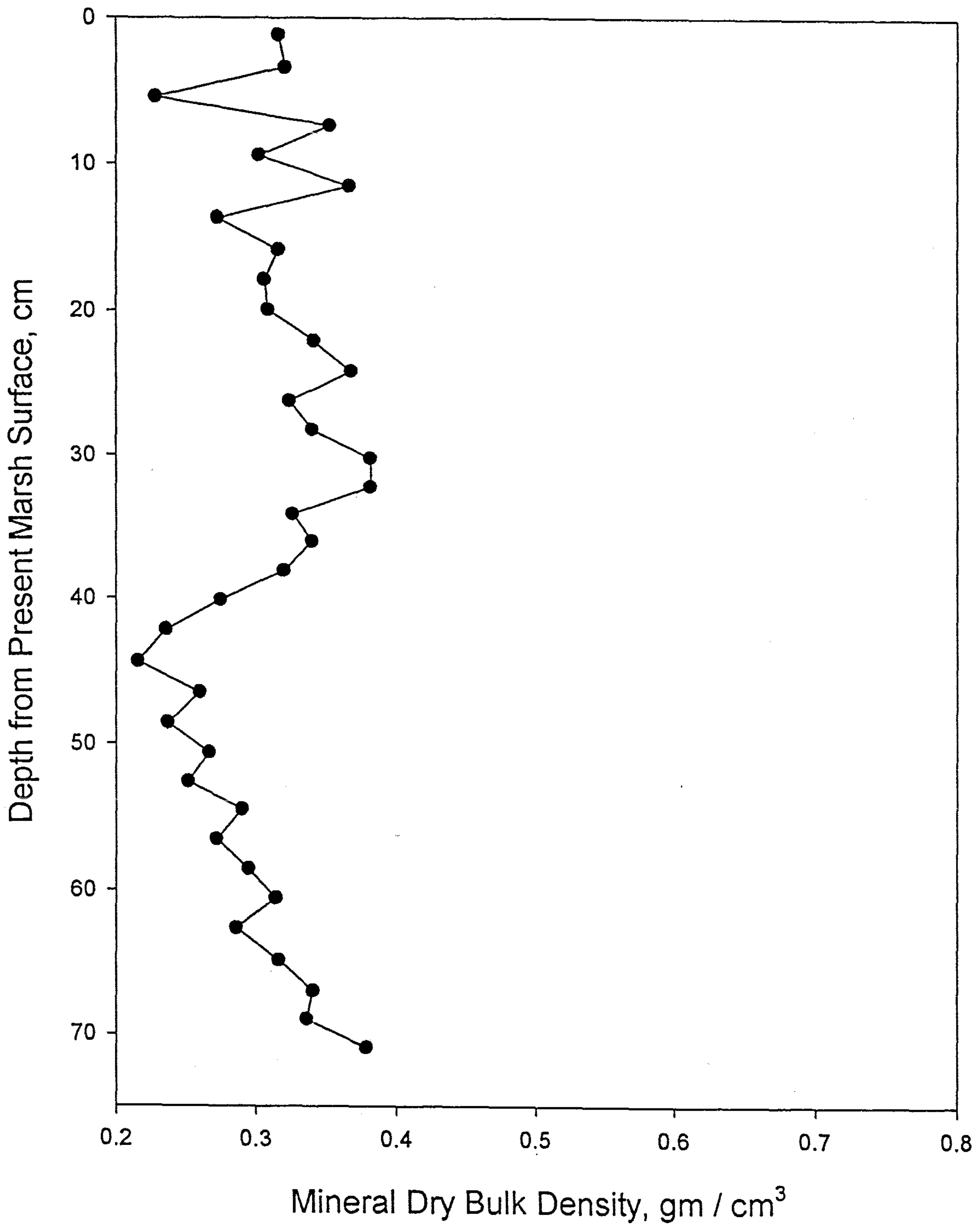


FIG. 5, MINERAL DRY BULK DENSITIES OF CORE FROM STATION 2+100

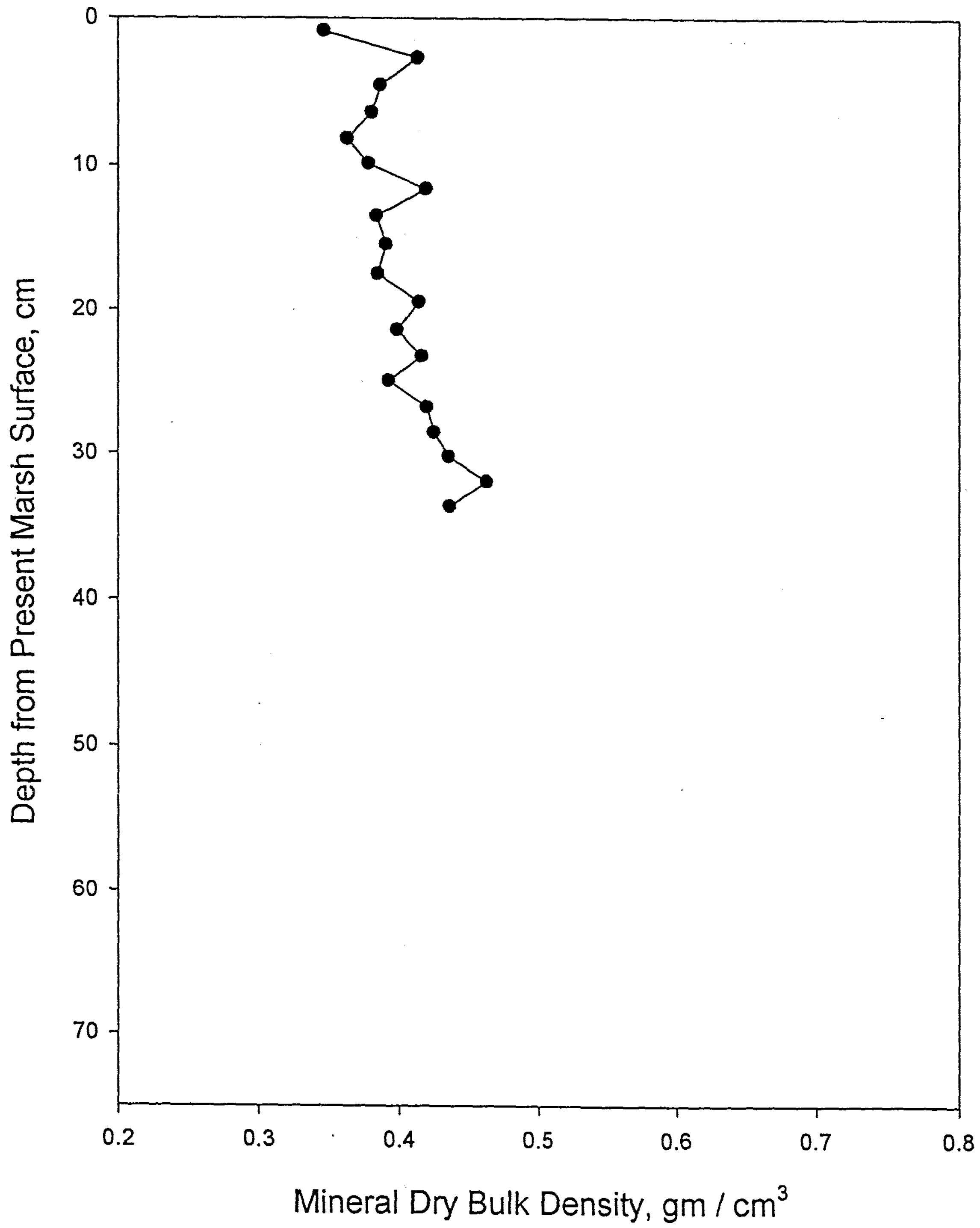


FIG. 6, MINERAL DRY BULK DENSITIES OF CORE FROM STATION 4+050

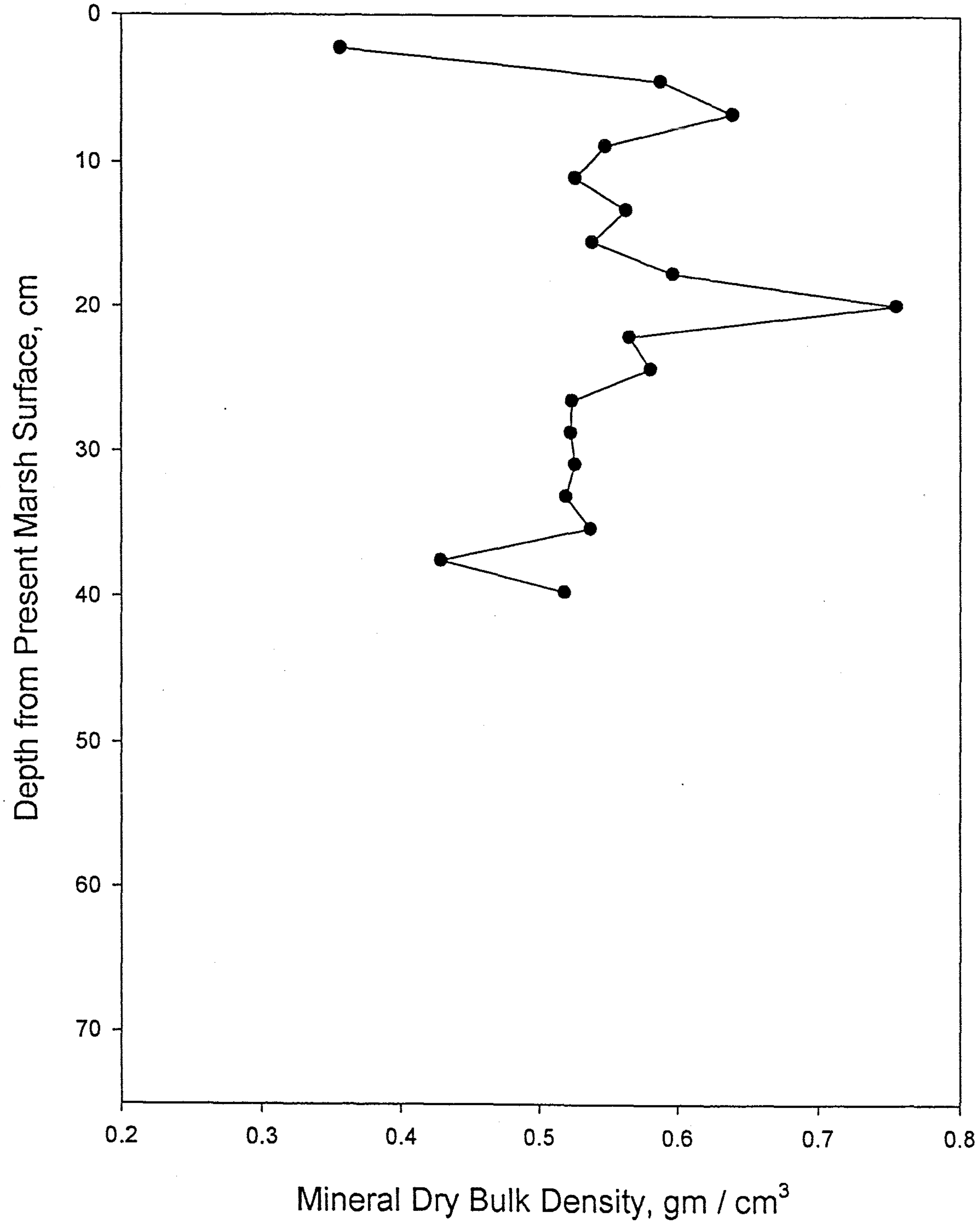


FIG. 7, MINERAL DRY BULK DENSITIES OF CORE FROM STATION 6+600

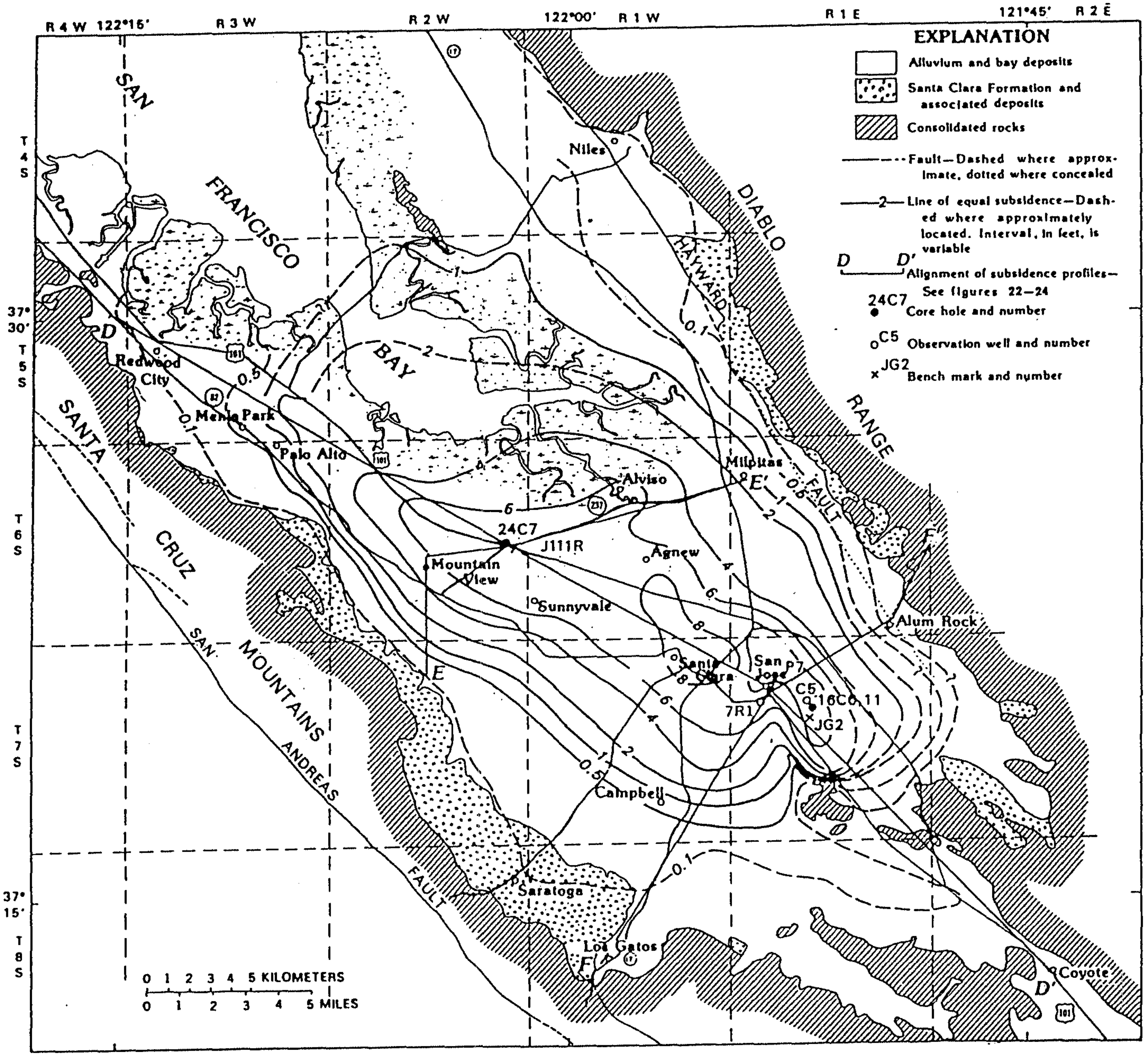


FIG. 8, LAND SUBSIDENCE FROM 1934 TO 1967
SANTA CLARA COUNTY, from Poland & Ireland

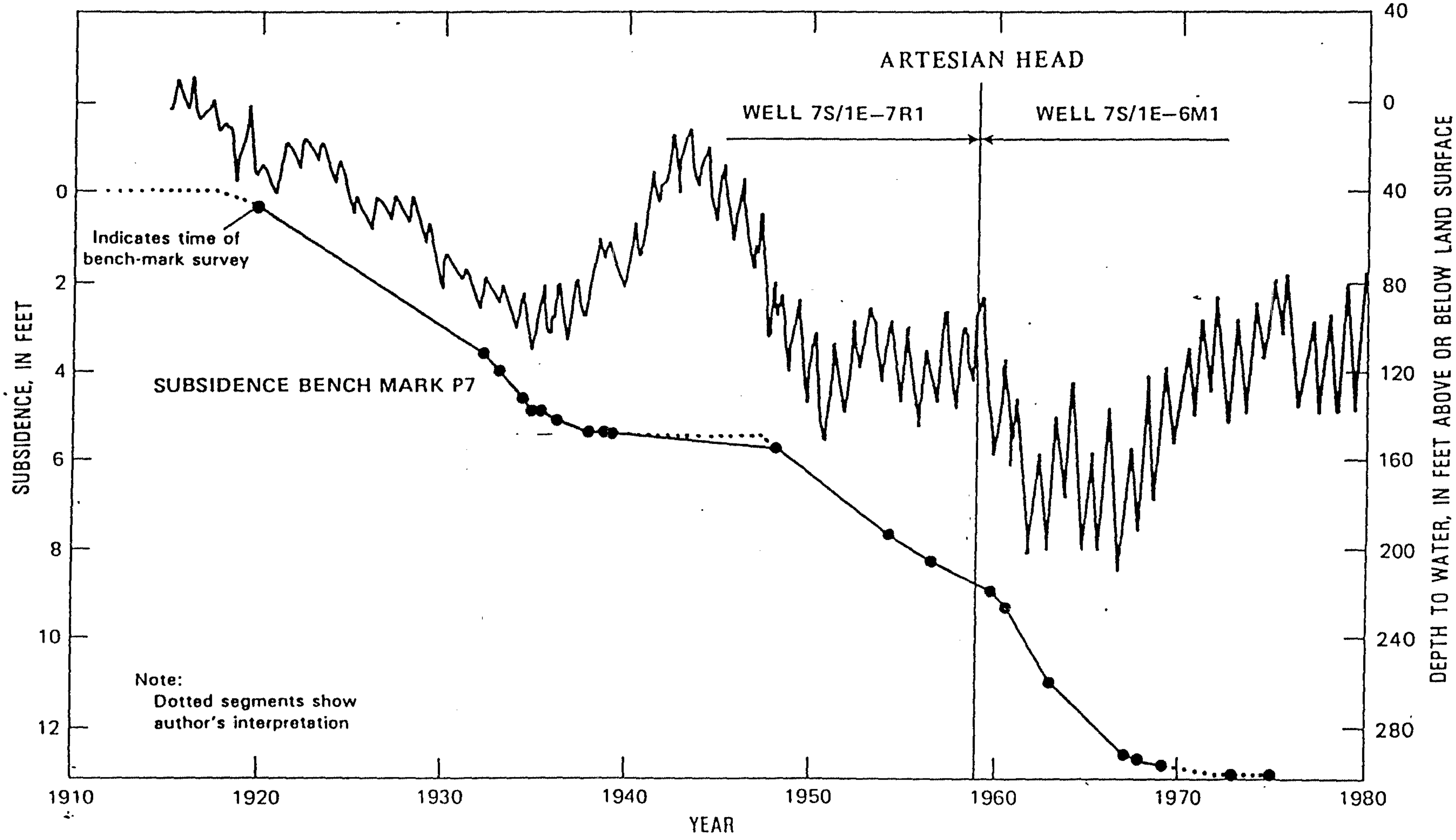
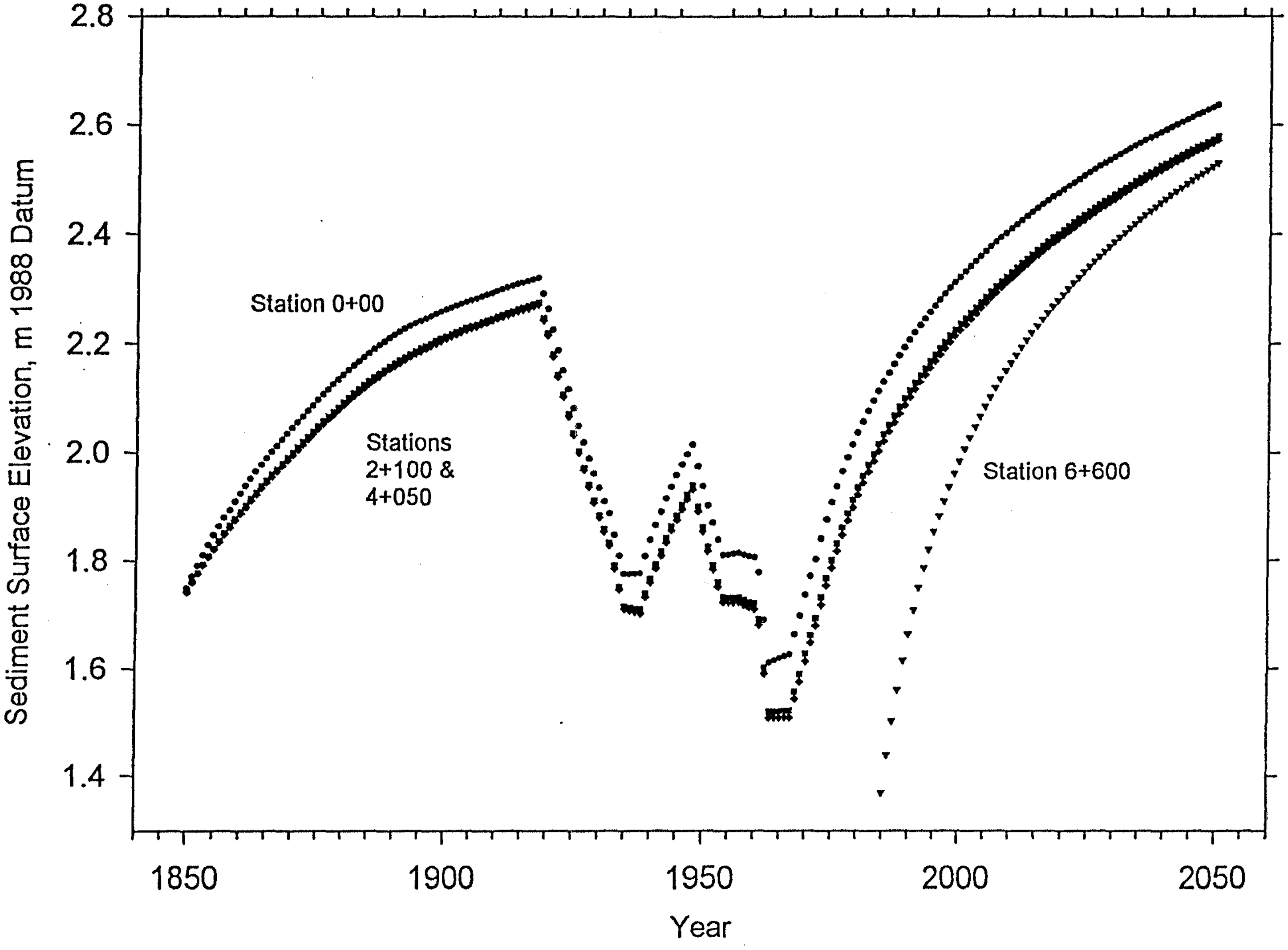


FIG. 9, ARTESIAN HEAD CHANGE AND LAND SUBSIDENCE, SAN JOSE, from Poland & Ireland



HISTORICAL AND PROJECTED ELEVATIONS AT MARSH STATIONS

APPENDIX

Basic Data

northwest hydraulic consultants inc.

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vancouver
edmonton
seattle

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west sacramento, california 95691-3430
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21 April, 1998

Dr. Ray Krone
R.B. Krone and Associates
PO Box 694
Davis, CA
95617

Subject: Guadalupe River Sedimentation Study - Bay Mud Samples

Dear Dr. Krone,

Please find enclosed the laboratory testing results for moisture, bulk density, and organic matter of the three bay mud samples we took earlier this year in February. Also enclosed are tables showing the depths of each sample and the ground surface elevation at each core site. Please note that all of these data are in **metric** units.

If you have any questions regarding these materials or require any additional information, please feel free to call at any time.

Yours Sincerely,
Northwest Hydraulic Consultants, Inc.

Rene F. Leclerc M.S.

Robert C. MacArthur, Ph.D., P.E.
Principal

Enclosures

RIVER ENGINEERING / HYDRAULIC MODEL TESTING / HYDRAULIC ANALYSIS AND DESIGN / COASTAL ENGINEERING
HYDROLOGY / SEDIMENTATION ENGINEERING / NUMERICAL MODELING / APPLIED RESEARCH / FORENSIC ENGINEERING

Core A			Core B			Core C		
Sample Number	Average Thickness (cm)	Depth (cm)	Sample Number	Average Thickness (cm)	Depth (cm)	Sample Number	Average Thickness (cm)	Depth (cm)
A1	1.75	1.75	B1	2.34	2.34	C1	1.93	1.93
A2	1.83	3.58	B2	2.04	4.39	C2	2.27	4.19
A3	1.85	5.43	B3	1.95	6.33	C3	2.21	6.40
A4	1.88	7.31	B4	2.04	8.38	C4	2.29	8.69
A5	1.69	9.00	B5	2.09	10.46	C5	2.11	10.80
A6	1.72	10.72	B6	2.13	12.59	C6	2.12	12.92
A7	1.79	12.51	B7	2.18	14.77	C7	1.87	14.79
A8	1.89	14.40	B8	2.10	16.87	C8	2.20	16.99
A9	2.04	16.44	B9	1.95	18.82	C9	1.72	18.72
A10	2.04	18.47	B10	2.20	21.02	C10	2.21	20.92
A11	1.92	20.39	B11	2.01	23.03	C11	2.20	23.12
A12	1.95	22.34	B12	2.14	25.17	C12	2.21	25.33
A13	1.75	24.09	B13	2.00	27.17	C13	1.93	27.26
A14	1.69	25.78	B14	2.05	29.22	C14	2.03	29.30
A15	1.98	27.76	B15	1.97	31.19	C15	1.95	31.25
A16	1.53	29.29	B16	2.09	33.28	C16	2.37	33.62
A17	1.82	31.11	B17	1.75	35.03	C17	2.09	35.71
A18	1.66	32.77	B18	2.01	37.04	C18	2.06	37.77
A19	1.70	34.47	B19	2.07	39.12	C19	1.91	39.68
			B20	1.99	41.10	C20	2.17	41.85
			B21	2.18	43.28	C21	2.24	44.09
			B22	2.14	45.42	C22	2.03	46.12
			B23	2.12	47.54	C23	2.27	48.39
			B24	2.02	49.56	C24	2.27	50.67
			B25	2.13	51.69	C25	2.15	52.82
			B26	1.83	53.52	C26	2.29	55.11
			B27	1.94	55.46	C27	2.23	57.34
			B28	2.11	57.57	C28	2.12	59.46
			B29	1.94	59.51	C29	2.19	61.65
			B30	2.01	61.52	C30	2.03	63.68
			B31	2.19	63.71			
			B32	2.26	65.97			
			B33	2.03	68.00			
			B34	1.90	69.90			
			B35	2.01	71.91			

SCVWD

Land Surveying and Mapping

Project: Guadalupe River Soil Boring Sites
Folder: 121
Date Processed: 4/13/98
Horizontal Datum: NAD83
Vertical Datum: NAVD88
Note* All units in Metric Measurements

Point#	sta. (m)	Northing	Easting	Elevation	
P-3	0 m	607794.181	1864945.274	2.291	— CORE C
P-2	2100 m	606048.177	1865810.633	2.191	— CORE B
P-1	4050 m	605305.382	1867399.010	2.202	— CORE A

↑

DISTANCE UPSTREAM OF MOUTH

s:\Guad-rv\Fld121\SoilBoring.xls

Sunland Analytical Lab, Inc.
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 02/12/98
Date Submitted 02/02/98

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 18679. Your purchase order number is .
Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41328	CORE A	1 (TOP)	59.00	1.47	15.4	WATER +
41329	CORE A	2	53.13	1.31	13.5	ROOTS
41330	CORE A	3	53.93	1.28	14.0	ROOTS
41331	CORE A	4	54.72	1.29	13.9	ROOTS
41332	CORE A	5	53.91	1.25	15.2	ROOTS
41333	CORE A	6	55.93	1.32	13.5	ROOTS
41334	CORE A	7	54.24	1.38	13.5	ROOTS
41335	CORE A	8	53.58	1.24	13.6	ROOTS
41336	CORE A	9	52.17	1.19	13.2	
41337	CORE A	10	53.38	1.23	13.5	
41338	CORE A	11	54.34	1.32	12.4	
41339	CORE A	12	54.13	1.32	13.8	
41340	CORE A	13	53.19	1.31	13.2	DRK COLOR
41341	CORE A	14	54.58	1.29	13.1	DRK COLOR

Sunland Analytical Lab, Inc.
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Date Reported 02/12/98
Date Submitted 02/02/98

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 18680. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41342	CORE A	15	52.95	1.32	13.4	DRK COLOR
41343	CORE A	16	53.44	1.34	13.0	DRK COLOR
41344	CORE A	17	52.77	1.34	12.9	DRK COLOR
41345	CORE A	18	54.04	1.48	12.8	DRK COLOR
41346	CORE A	19	55.03	1.40	11.9	DRK COLOR

Sunland Analytical Lab, Inc.
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Date Reported 02/12/98
Date Submitted 02/02/98

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 18681. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41347	CORE B	1 (TOP)	58.87	1.21	13.0	H20,ROOTS
41348	CORE B	2	56.35	1.14	13.6	
41349	CORE B	3	56.43	1.07	20.3	
41350	CORE B	4	55.47	1.19	13.0	
41351	CORE B	5	56.56	1.20	16.3	ROOTS
41352	CORE B	6	55.00	1.17	11.8	ROOTS
41353	CORE B	7	55.10	1.14	19.1	ROOTS
41354	CORE B	8	54.93	1.09	14.2	ROOTS
41355	CORE B	9	54.49	1.10	15.8	
41356	CORE B	10	55.76	1.11	14.5	ROOTS
41357	CORE B	11	55.77	1.25	15.0	
41358	CORE B	12	53.55	1.21	14.2	RTS, DRKER
41359	CORE B	13	54.62	1.14	15.1	ROOTS
41360	CORE B	14	54.27	1.17	14.8	

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Date Reported 02/12/98
Date Submitted 02/02/98

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 18682. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41361	CORE B	15	53.47	1.26	14.4	
41362	CORE B	16	54.19	1.25	13.4	ROOTS
41363	CORE B	17	55.01	1.12	14.0	
41364	CORE B	18	54.85	1.23	15.6	
41365	CORE B	19	55.99	1.19	15.2	ROOTS
41366	CORE B	20	57.66	1.12	15.8	ROOTS
41367	CORE B	21	58.79	1.05	16.7	DARKER
41368	CORE B	22	59.87	1.03	17.1	ROOTS
41369	CORE B	23	58.56	1.12	16.2	ROOTS
41370	CORE B	24	58.45	1.04	16.7	ROOTS
41371	CORE B	25	57.48	1.10	16.3	RTS,H20+
41372	CORE B	26	59.79	1.18	16.8	
41373	CORE B	27	56.84	1.19	16.8	
41374	CORE B	28	57.17	1.16	17.4	

Sunland Analytical Lab, Inc.
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Date Reported 02/12/98
Date Submitted 02/02/98

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 18683. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41375	CORE B	29	55.99	1.15	16.4	
41376	CORE B	30	57.18	1.23	15.3	ROOTS
41377	CORE B	31	57.36	1.12	15.1	
41378	CORE B	32	56.75	1.16	14.0	ROOTS
41379	CORE B	33	57.23	1.24	13.3	LTR,H20+
41380	CORE B	34	56.95	1.19	12.8	
41381	CORE B	35	54.91	1.23	12.4	

Sunland Analytical Lab, Inc.
11353 Pyrites Way, Suite 4
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Date Reported 02/12/98
Date Submitted 02/02/98

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 18684. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41382	CORE C	1 (TOP)	53.75	1.17	17.7	ROOTS,H20
41383	CORE C	2	46.67	1.03	15.8	
41384	CORE C	3	46.13	1.05	16.5	ROOTS
41385	CORE C	4	47.22	1.08	16.8	ROOTS
41386	CORE C	5	48.06	1.14	14.8	
41387	CORE C	6	47.43	1.32	15.6	
41388	CORE C	7	48.93	1.13	14.8	
41389	CORE C	8	49.37	1.19	15.4	ROOTS
41390	CORE C	9	49.48	1.23	14.9	
41391	CORE C	10	49.59	1.19	15.9	ROOTS
41392	CORE C	11	48.95	1.30	14.5	
41393	CORE C	12	50.39	1.21	14.9	
41394	CORE C	13	50.36	1.25	16.4	RTS,DRKE
41395	CORE C	14	52.33	1.24	16.9	

Sunland Analytical Lab, Inc.
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 02/12/98
Date Submitted 02/02/98

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 28685. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample #	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41396	CORE C	15	53.67	1.15	16.0	ROOTS
41397	CORE C	16	50.26	1.26	17.0	ROOTS
41398	CORE C	17	53.46	1.25	16.5	
41399	CORE C	18	52.94	1.27	15.7	ROOTS
41400	CORE C	19	54.27	1.14	15.0	RTS,H20+
41401	CORE C	20	52.22	1.21	15.1	ROOTS
41402	CORE C	21	53.75	1.20	16.3	RTS,DRKER
41403	CORE C	22	54.02	1.22	15.7	ROOTS
41404	CORE C	23	55.56	1.15	16.7	ROOTS
41405	CORE C	24	53.77	1.19	17.2	ROOTS
41406	CORE C	25	56.01	1.09	15.4	ROOTS
41407	CORE C	26	56.51	1.04	17.9	ROOTS
41408	CORE C	27	54.15	1.12	16.3	ROOTS
41409	CORE C	28	55.85	1.02	16.7	ROOTS

Sunland Analytical Lab, Inc.
11353 Pyrites Way, Suite 4
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(916) 852-8557

Date Reported 02/12/98
Date Submitted 02/02/98

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 38685. Your purchase order number is .

Thank you for your business.

SUN #	Sample Describ	Sample \#	Moisture %	Bulk Density g/cc	Organic Matter %	Notes
41410	CORE C	29	54.68	1.21	15.8	ROOTS
41411	CORE C	30	54.84	1.28	16.2	ROOTS

Sunland Analytical Lab, Inc.
11353 Pyrites Way, Suite 4
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(916) 852-8557

Date Reported 08/28/1997
Date Submitted 08/22/1997

To: Rene Leclerc
Northwest Hydraulic Consultants
3950 Industrial Blvd #100
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 8092. Your purchase order number is .
Thank you for your business.

SUN #	Sample Describ	Sample #	Depth (cm)	Moisture %	Bulk Density g/cc	Organic Matter %
39890	CORE 6+600	13	28.68	37.42	1.11	14.1
39891	CORE 6+600	14	30.88	37.60	1.15	15.3
39892	CORE 6+600	15	33.08	37.23	1.11	14.6
39893	CORE 6+600	16	35.29	38.93	1.28	17.7
39894	CORE 6+600	17	37.49	43.71	1.15	17.4
39895	CORE 6+600	18	39.7	42.68	1.19	12.2